TETRA TECH EC, INC.

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October 26, 2009

California Energy Commission
Docket No. 09-AFC-8
1516 9th St.
Sacramento, CA 95814

Genesis Solar Energy Project - Docket Number 09-AFC-8

Docket Clerk:

Included with this letter is a hard copy and CD-ROM of the following reports:

- E.2- *Geophysical Shear Wave Investigation at Ford Dry Lake near Blythe in Riverside County, California*
- E.3- *Preliminary Geotechnical and Geologic Hazards Investigation for Genesis Solar Energy Project Chuckwalla Valley Riverside County, California*

These reports are insertions to the Genesis Solar Energy Project Application for Certification, Appendix E, originally submitted on August 31st, 2009.

All information contained in these reports is believed to be accurate and true.

Sincerely,

Tricia Bernhardt
Project Manager/Tetra Tech EC

cc: Mike Monasmith /CEC Project Manager

J R ASSOCIATES

Engineering Geophysics
1886 Emory Street
San Jose, CA 95126
(408) 293-7390

GEOPHYSICAL SHEAR WAVE INVESTIGATION AT FORD DRY LAKE NEAR BLYTHE IN RIVERSIDE COUNTY, CALIFORNIA

September 21, 2009

for

WorleyParsons Group, Incorporated
2330 E Bidwell Street, Suite 150
Folsom, CA 95630

by

James Rezowalli
California Registered Geophysicist, GP-921

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I INTRODUCTION

This report presents the results of a geophysical shear (S-) wave investigation performed north of Ford Dry Lake near Blythe in Riverside County, California. The investigation was performed for WorleyParsons Group, Incorporated, by J R Associates. The objectives of the investigation were:

> Conduct a downhole shear wave test at the shallow observation well installed at the test well cluster to look for low shear wave velocities that are an indication of weak soil zones.
>
> Collect shear wave velocity profiles at three locations using the Multichannel Analysis of Surface Waves (MASW) method. Compare MASW results to downhole shear wave data. Look for low velocity shear wave zones indicative of weak soils under the three MASW traverses.

James Rezowalli, Principal Geophysicist, Garret Rhett, Technician, and Jeff Spackman, Technician, of J R Associates performed the field work in September of 2009.

A. <u>Site Conditions</u>

The area of interest is just north of Ford Dry Lake approximately 20 miles west of Blythe, California (Drawing 1). The site consists of dry flat desert and dry lake bed. Lithologic logs from test wells at the site indicate the upper 75 feet of soil is a younger alluvium containing a mixture of sands, silts, and clays. The water table at the site is approximately 75 feet below grade.

Genesis Solar LLP proposes to develop a power plant at the site. Information on compressible or liquefiable soils was needed for the project. Studies have shown a relationship between shear wave velocities and liquefaction resistance of soils[1]. In general soils with low shear wave velocities are more prone to liquefaction than soils with higher shear wave velocities. Because most of the site is only accessible by foot and motor vehicles are prohibited, conventional methods for determining soil strength, such as a cone penetrometer or a standard penetration test, were not allowed. The MASW method of collecting shear wave velocity profiles was chosen because it could be performed on foot in areas presently inaccessible to drill rigs. Shear wave data were also collected in an existing observation well.

[1]Andrus, R.D. and Stokoe, K.H. (2000), "Liquefaction Resistance of Soils From Shear Wave Velocity." *Journal of Geotechnical and Geoenvironmental Engineering*, Vol 126, No 11, November 2000, 1015-1025.

II METHODOLOGY

We used two geophysical methods in our investigation, downhole compressional (P-) and shear (S-) wave measurements and the multichannel analysis of surface wave method (MASW). Drawing 2 illustrates the two methods. The downhole method involves creating P- and S-waves on the surface and measuring their travel times to a receiver in a borehole. From a graph of travel times versus depth, P- and S- wave velocities for the soil adjacent to the borehole are calculated creating a one-dimensional velocity profile. The MASW method involves measuring the dispersion of a surface wave created at one end of a string of receivers. From the dispersion data a one dimensional S-wave velocity profile is calculated. By collecting several profiles along a traverse, a two-dimensional shear wave profile can be created.

A. Downhole Field Procedures and Instrumentation

Two downhole P- and S-wave profiles were collected in the shallow observation well at the test well cluster (Drawing 3). We began data collection by installing a P-wave and an S-wave source on the ground near the borehole. The P-wave source consisted of a 12-pound sledge hammer striking an aluminum plate. The S-wave source consisted of a 4x4 wooden beam laying on its side on the ground. We drove a truck onto the beam to hold it in place. One end of the beam was struck with the sledge hammer to create an S-wave. We could change the polarization of the S-wave by striking the other end of the beam. S-waves are picked from a seismograph recording by looking for a standout in amplitude and the polarity reversal in the recorded wave forms.

At the start of a test a triaxial geophone was lowered to the bottom of a borehole and locked to a borehole wall. We then generated a P-wave and a pair of S-waves on the ground surface and recorded their arrivals at the geophone. The S-wave pair consisted of a forward polarized wave and a reversed polarized wave. We then raised the geophone 5 feet and collected another set of waves. This process was repeated until the geophone was 5 feet from the ground surface. We collected two sets of data, one with the sources ten feet from the borehole and the other with the sources fifteen feet from the borehole.

A Litton LRS-1023 triaxial geophone was lowered into the borehole to detect the seismic signals. A cable connected the geophones to a Geometrics Geode seismograph which in turn was connected to a personal computer. The computer filtered, stacked, and recorded the signals. Stacking (adding) signals from multiple hammer blows at the same source point improves the signal to noise ratios of the recordings. Typically four recordings at each geophone depth and source were stacked.

Data reduction began by picking the arrival times from the seismograph recordings. An arrival time is the time a wave spent traveling from a source point to the geophone. The waves were assumed to travel in a straight line from the source to the triaxial geophone. The arrival times versus depths were plotted and the P- and S-wave velocities were calculated from the plot. We calculated small strain values of Poisson's ratio and shear modulus from the averaged P- and S-wave velocities. A unit weight of 110 pounds per cubic foot was assumed for the shear modulus calculation.

B. <u>MASW Field Procedures and Instrumentation</u>

MASW data were collected along a test line adjacent to the well cluster and along three 294-foot profile lines on the eastern side of the site (Drawing 3). Data were collect along the test line to establish the optimum shot point offset and to compare the MASW and downhole results.

MASW data collection began by placing the plate 30 feet from the end of a string of 24 geophones. The geophones were spaced three feet apart. Surface waves were created by striking an aluminum plate and the waves were recorded. Once a multichannel record was collected, the plate and geophone array were advanced 15 feet along the line and the process was repeated. A total of fourteen records were collected along each shear wave line.

Data were collected using 4.5-Hz geophones connected to a Geometrics Geode seismograph which in turn was connected to a personal computer. The computer filtered, stacked, and recorded the signals. Stacking (adding) signals from multiple hammer blows at the same source point improves the signal to noise ratios of the recordings. Typically four recordings were stacked.

The program Surfseis developed by the Kansas Geological Survey was used to process the seismic records into S-wave profiles. From each seismic recording a fundamental-mode dispersion curve was extracted. The dispersion curve is related to the shear wave velocities of the different wave lengths contained in the surface wave. Longer wave lengths are related to the S-wave velocity of deeper soils and shorter wave lengths are related to the S-wave velocities of near surface soils. The dispersion curves are inverted into a series of one-dimensional S-wave velocity profiles that are concatenated together into a two-dimensional profile. More information of the MASW can be found at the Kansas Geological Survey's web site at www.kgs.edu/software/surfseis/.

III RESULTS

A. Downhole Results

Drawing 4 and Table 1 give the results of the two downhole P- and S-wave profiles collected in the test well. The two graphs show plots of P-and S-wave arrival times versus depth. Drawing 2 also shows the average P- and S-wave velocity for the upper 75 feet of soil along with the average small-strain shear modulus and small strain Poisson's ratio. The unit weight of the soil was assumed to be 110 pounds per cubic foot for calculating the shear modulus.

Table 1. Summary of Downhole Results

Layer Number	Depth (feet)	S-wave (fps)	P-wave (fps)
1	0 to 10	1100 to 1200	1900 to 2100
2	10 to 25	1300	2700 to 2800
3	25 to 40	800 to 850	1450 to 1500
4	40+	1000 to 1100	2400 to 3400

The data indicated four layers that were distinguished by their P- and S-wave velocities. Typically P- and S-wave velocities increase with depth. At the well site the second layer had higher S-wave velocities than the third layer and had the greatest S-wave velocity of all four layers. The higher S-wave velocity in the second layer may be due to weak cementing.

B. MASW Results

The results of the MASW data are shown on Drawing 5 and Table 2. Drawing 5 illustrates the S-wave velocity profiles collected along the four MASW lines and Table 2 shows the average S-wave velocities for each of the four seismic layers beneath each line along with an error estimate equal to one standard deviation.

Table 1. Average S-Wave Velocities for MASW Profiles

Line Number	Layer 1 S-wave (fps)	Layer 2 S-wave (fps)	Layer 3 S-wave (fps)	Layer 4 S-wave (fps)
Test Line	800	1650	700	1400
Sw-1	1000 ±240	1750 ±270	700 ±64	1450 ±150
Sw-2	850 ±77	1800 ±190	750 ±100	1200 ±150
Sw-3	1050 ±240	1600 ±270	750 ±73	1450 ±280

Layer	Depth (feet)
1	0 to 10
2	10 to 25
3	25 to 45
4	45+

The MASW data shows four seismic layers defined by their S-wave velocities (Drawing 5). Like the downhole data the MASW results indicate the second layer had a greater S-wave velocity than the third and had the highest S-wave velocity of all four layers. The higher velocities in the second layer may be from weak cementing.

Comparing the MASW data and the downhole data indicates the MASW tends to overestimate the velocities of the faster layers and to underestimate the velocities of the slower layers by about 20 percent. The S-wave velocities of layers 1 and 3, layers with low S-wave velocities, are probably not slower than the averages shown on Table 1. The S-wave velocities for layers 2 and 4, layers with high S-wave velocities, are probably not faster than the average shown on Table 1.

C. Near Surface Refraction Results

Along with the MASW data we collected a short refraction line at each shear wave profile. The results of the refraction lines are shown in Drawing 6. The refraction data indicated two layers in the upper 20 feet of soil. The first layer is only a few feet thick and probably consists of loose surface soils. The second layer had a higher P-wave velocity and consists of denser soils. The relatively high P-wave velocities found along lines Sw-1 and Sw-3 indicate a possible caliche layer.

D. Compressibility and Liquefaction

The S-wave velocities of the third seismic layer indicate a layer of soil that is likely to be weaker than the layers above and below it. The S-wave velocities measured for the third seismic layer at a depth ranging from 25 to 45 feet varied from 700 to 850 fps and were considerably slower than the S-wave velocities measured at other depths. We recommend testing this zone further with standard geotechnical methods.

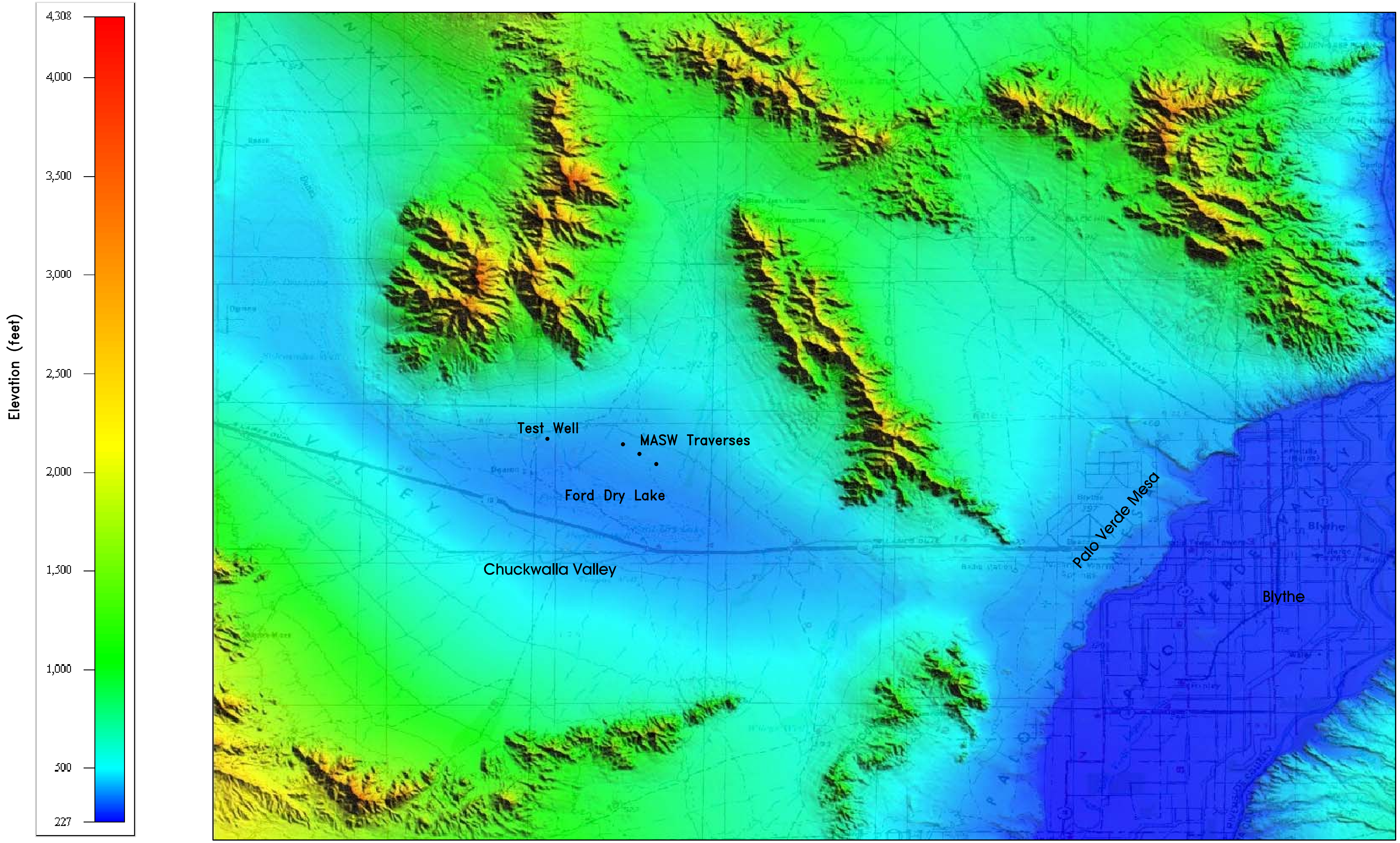
E. Summary

S-wave data were collected at four locations at the Ford Dry Lake site using two seismic methods (Drawing 3). Downhole shear wave data were collected at an observation well and shear wave velocity profiles were collected along four traverses using the multichannel analysis of surface waves method. Four seismic layers were found in the upper 75 feet of soil. The layers were distinguished by their P- and S-wave velocities (Drawings 4 and 5). The first layer was up to 10 feet thick and probably consisted of loose near surface soils. The second layer was between 10 and 25 feet below the surface. It probably consisted of denser sands, silts, and clays, possible lightly cemented, and may include caliche under lines Sw-1 and Sw-3. The third layer was from 25 to 45 feet below the ground surface. It was distinguished by P- and S-wave velocities that were slower than the soils above or below. The lower P- and S-wave velocities indicate the third layer was probably weaker than the layers above and below. We recommend testing the third layer further with conventional geotechnical methods such as a cone penetrometer or a standard penetration test. The fourth seismic layer probably consisted of denser sands, silts, and clays.

F. Limitations

Seismic layers do not always correspond directly to lithologic changes that might be found in borehole or trenching data. A seismic layer is an interface between materials with different seismic wave velocities. Factors such as weathering, cementation, induration, and saturation as well as lithologic changes can create changes in seismic velocities. Also, there can be lithologic changes without velocity changes. However, our field experience indicates that seismic layers often correspond to changes in lithology, cementation, or saturation to within ±20% of the depth to the interface.

IV DRAWINGS



Explanation:

Shear Wave Velocity Test Location

Elevation data from United States Elevation Data, NED, 30m Resolution

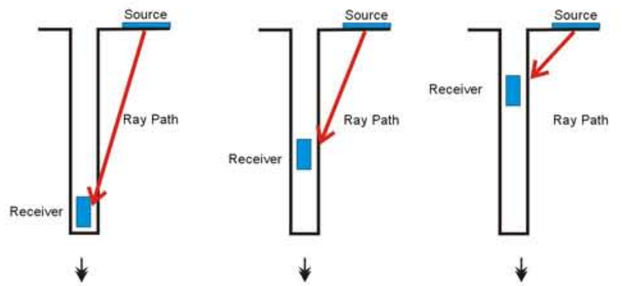
Vicinity and Elevation Map
Genesis Project Shear Wave Investigation
Riverside County California

SCALE: 1" = 4 Miles		DRAWN BY: J.J.R.
DATE: 9-17-2009	JOB NUMBER: 129-263-09	REVISED:

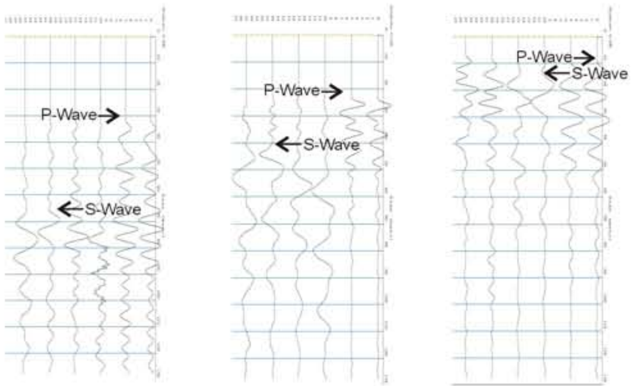
J R Associates Civil and Environmental Geophysics
1886 Emory Street, San Jose, CA (408) 293-7390

DRAWING NUMBER: 1

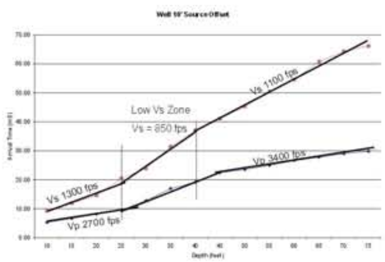
Field Set Up



Field Setup

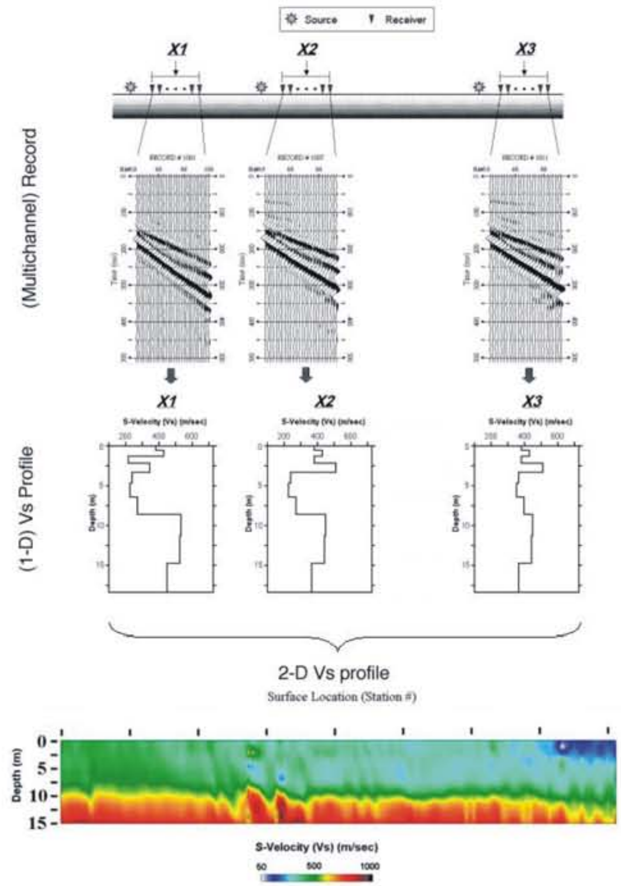


Multichannel Records



Arrival Time Graph

Field Set Up



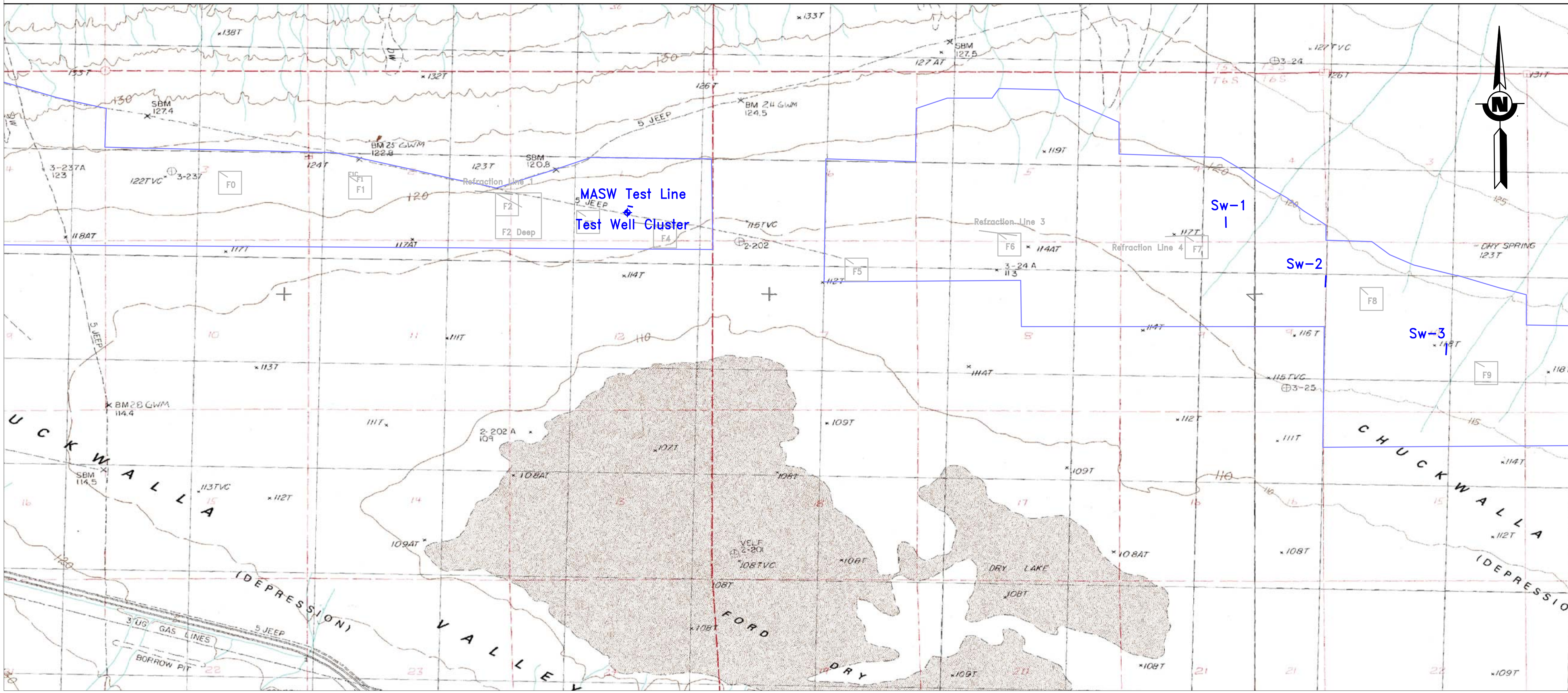
MASW graphic from the
Kansas Geological Survey Website

Downhole and MASW Shear Wave Techniques
Genesis Project Shear Wave Investigation
Riverside County, California

SCALE: No Scale			DRAWN BY: J.J.R.
DATE: 9-17-2009	JOB NUMBER:	129-263-09	REVISED:

J R ASSOCIATES Civil and Environmental Geophysics
1886 Emory Street, San Jose, CA (408) 293-7390

DRAWING NUMBER: 2



Explanation:

Sw-1 Shear Wave Velocity Profile

Downhole Shear Wave Measurements

0 0.5 1 1.5
Miles

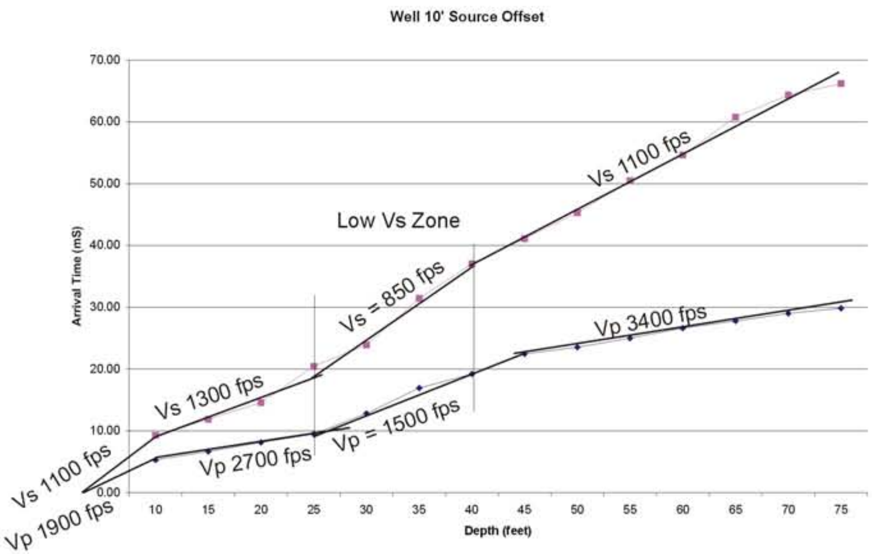
Shear Wave Profile Lines
Genesis Project Shear Wave Investigation
Riverside County California

SCALE: 1" = 0.5 Miles
DRAWN BY: J.J.R.
DATE: 9-17-2009
JOB NUMBER: 129-263-09
REVISED:

J R Associates Civil and Environmental Geophysics
1886 Emory Street, San Jose, CA (408) 293-7390

DRAWING NUMBER: 3

Downhole Shear Wave Measurements

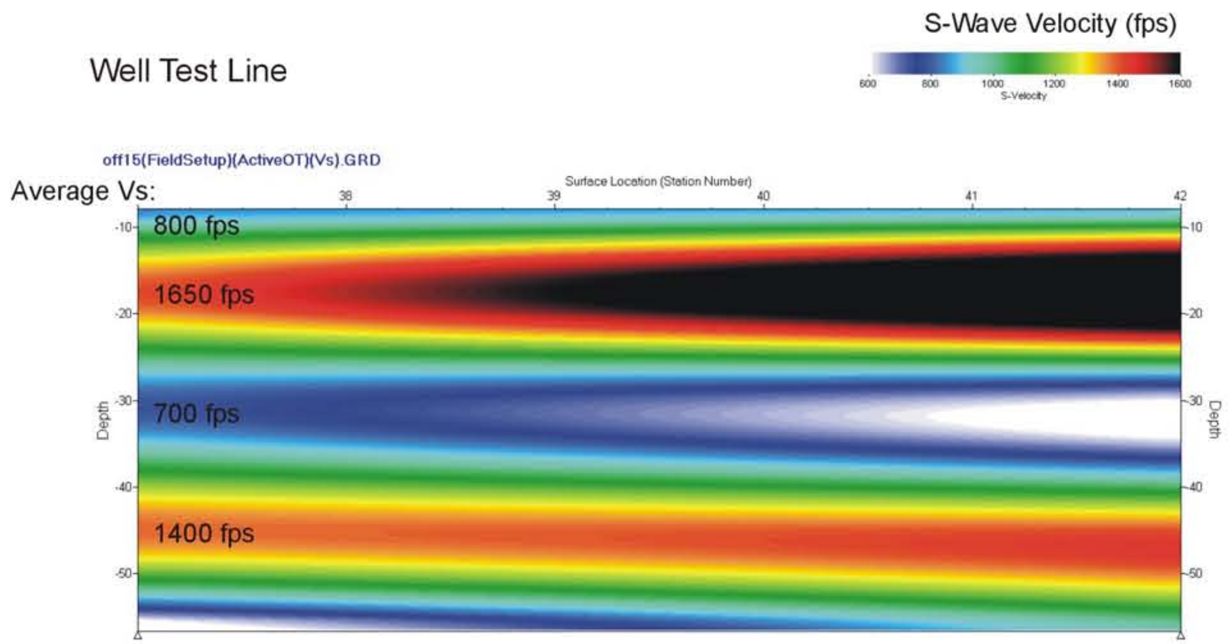


Average Vp = 2300 fps
Average Vs = 1100 fps
Average Dynamic Shear Modulus $\rho Vs^2 = 2.9 \times 10^4$ psi
Average Dynamic Poisson's Ratio = .35
Unit Weight Assumed to = 110 lb/ft^3

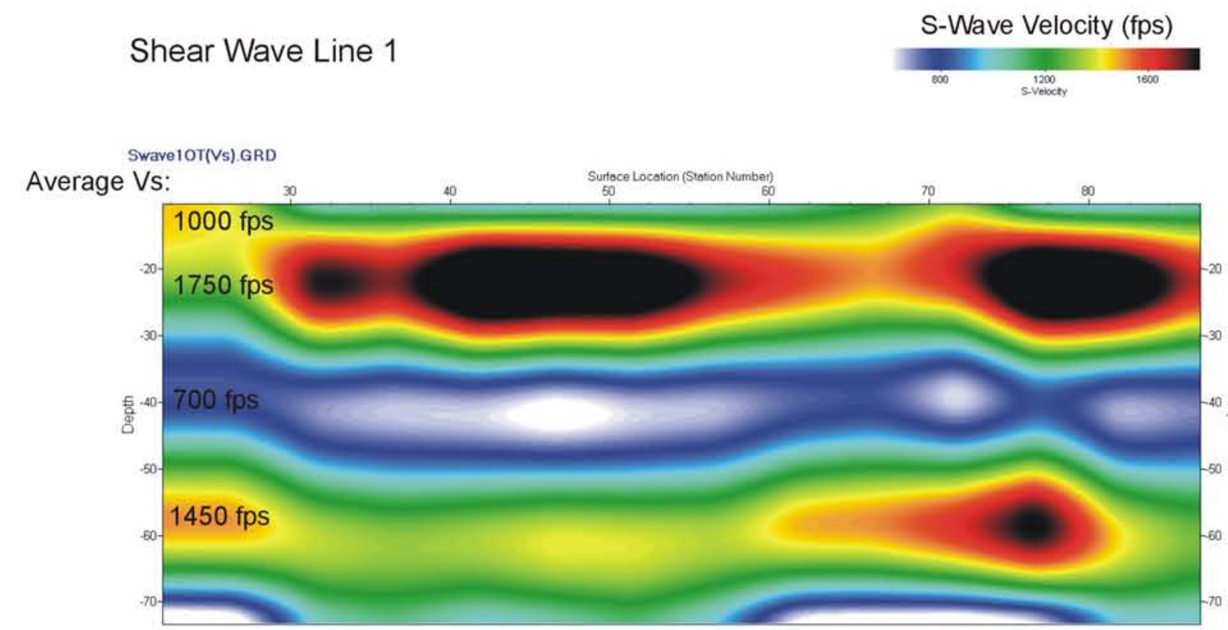
Average Vp = 2200 fps
Average Vs = 1000 fps
Average Dynamic Shear Modulus $\rho Vs^2 = 2.4 \times 10^4$ psi
Average Dynamic Poisson's Ratio = .37
Unit Weight Assumed to = 110 lb/ft^3

Downhole P- and S-wave Arrival Times Genesis Project Shear Wave Investigation Riverside County, California		
SCALE: See Diagrams		DRAWN BY: J.J.R.
DATE: 9-17-2009	JOB NUMBER: 129-263-09	REVISED:
J R ASSOCIATES Civil and Environmental Geophysics 1886 Emory Street, San Jose, CA (408) 293-7390		
		DRAWING NUMBER: 4

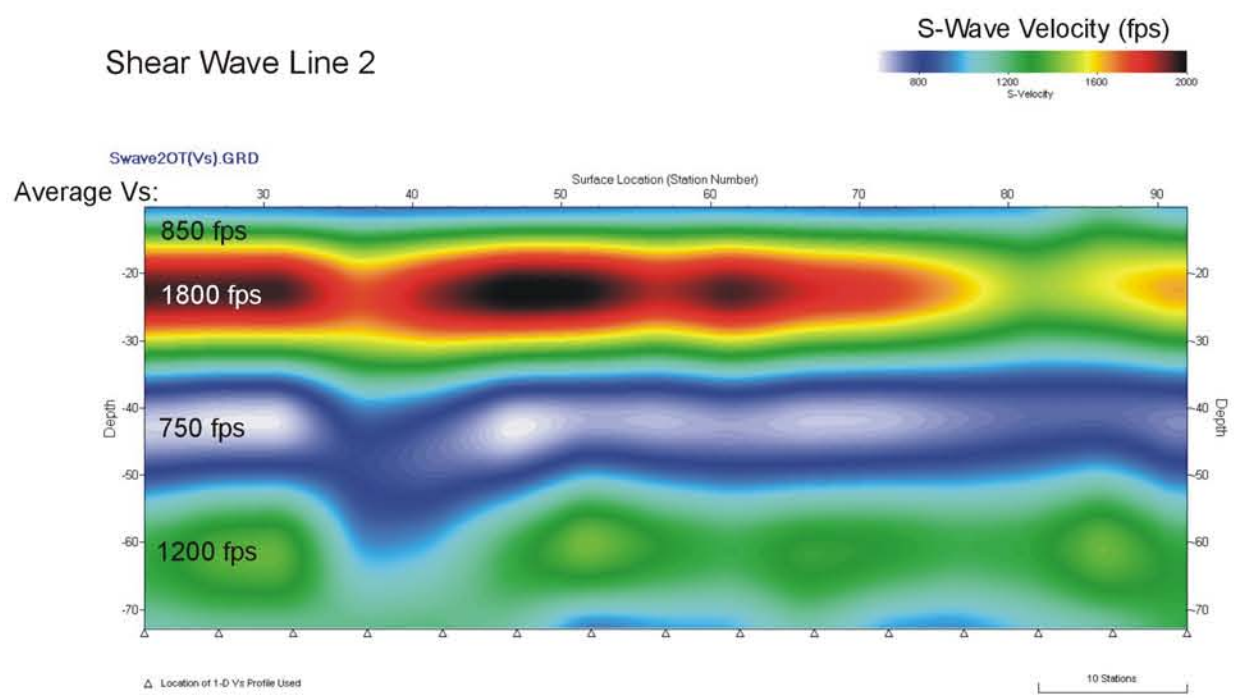
Well Test Line



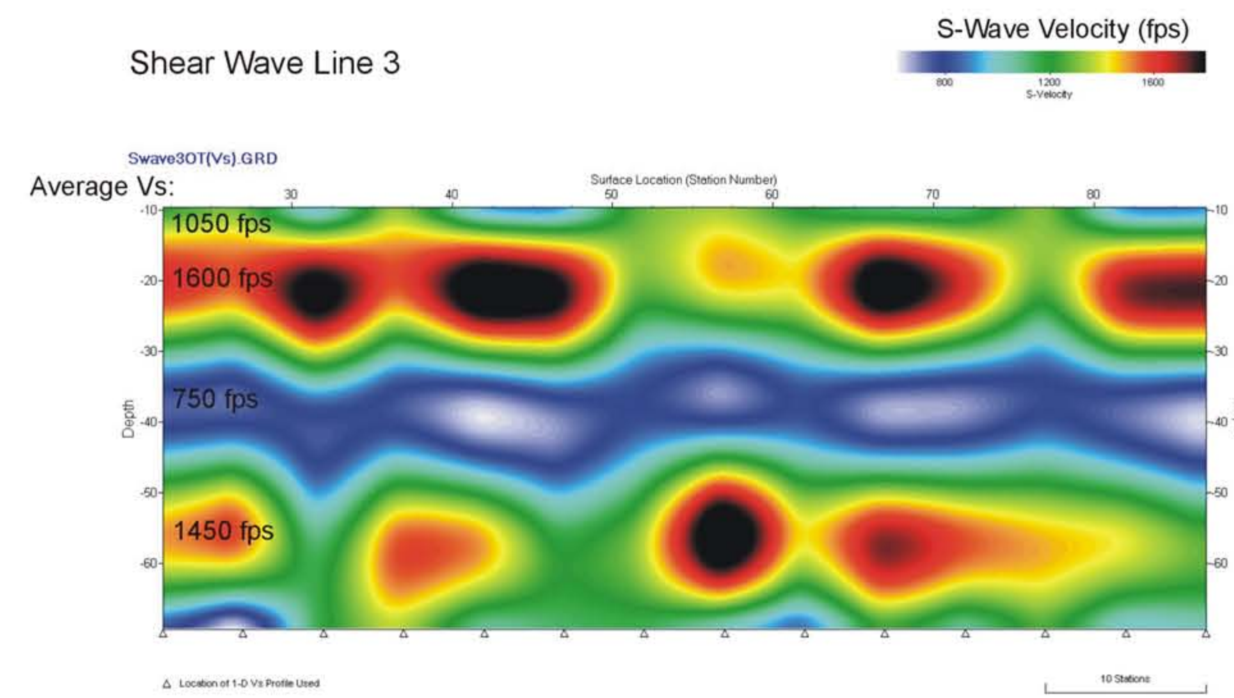
Shear Wave Line 1



Shear Wave Line 2



Shear Wave Line 3



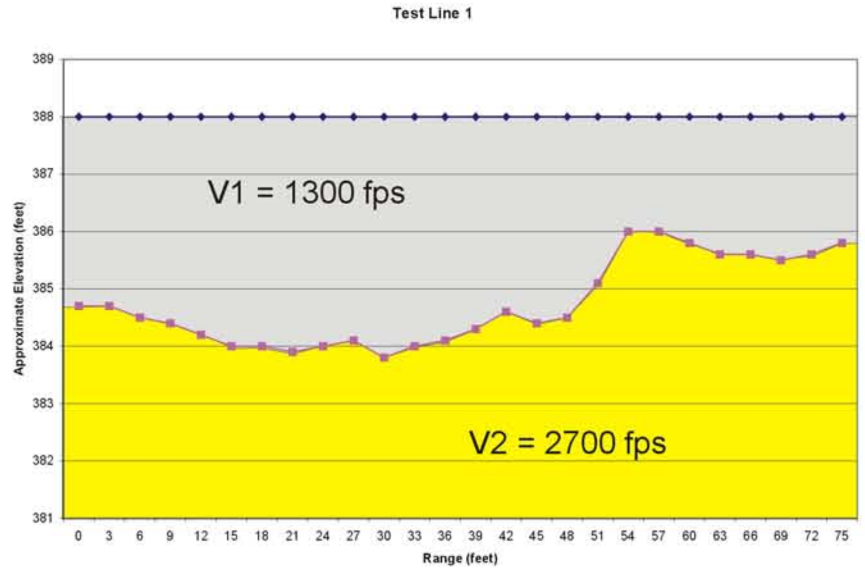
Shear Wave Velocity Profiles
Genesis Project Geophysical Investigation
Riverside County, California

SCALE: See Diagrams			DRAWN BY: J.J.R.
DATE: 9-17-2009	JOB NUMBER: 129-263-09		REVISED:

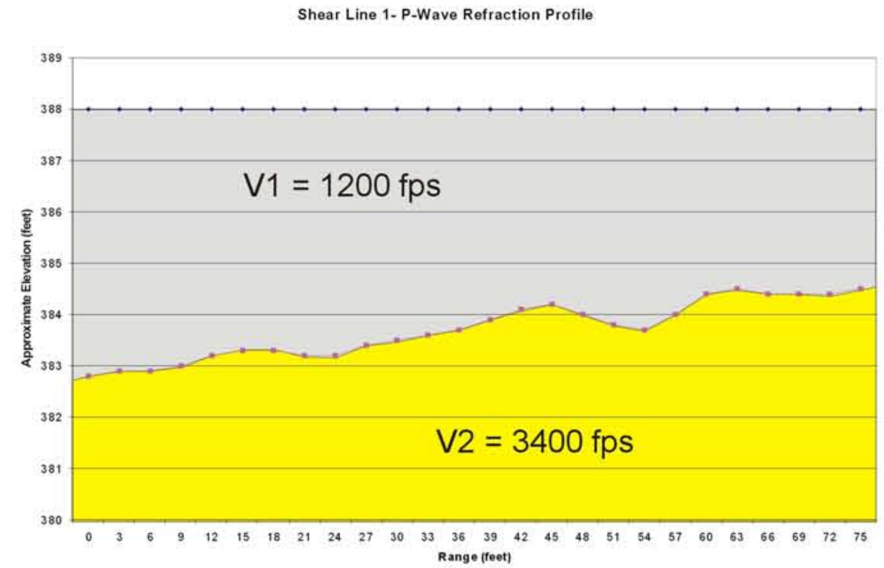
J R ASSOCIATES Civil and Environmental Geophysics
1886 Emory Street, San Jose, CA (408) 293-7390

DRAWING NUMBER: 5

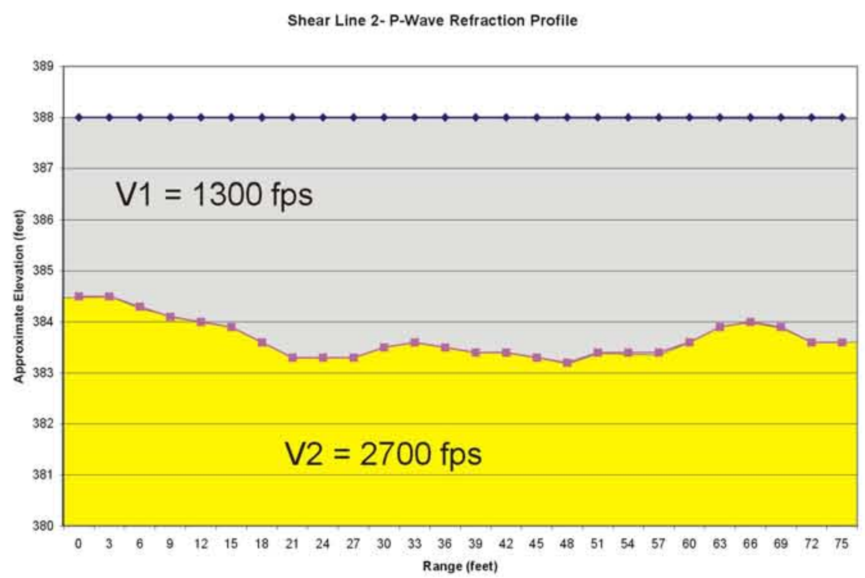
Refraction Line Near Test Well



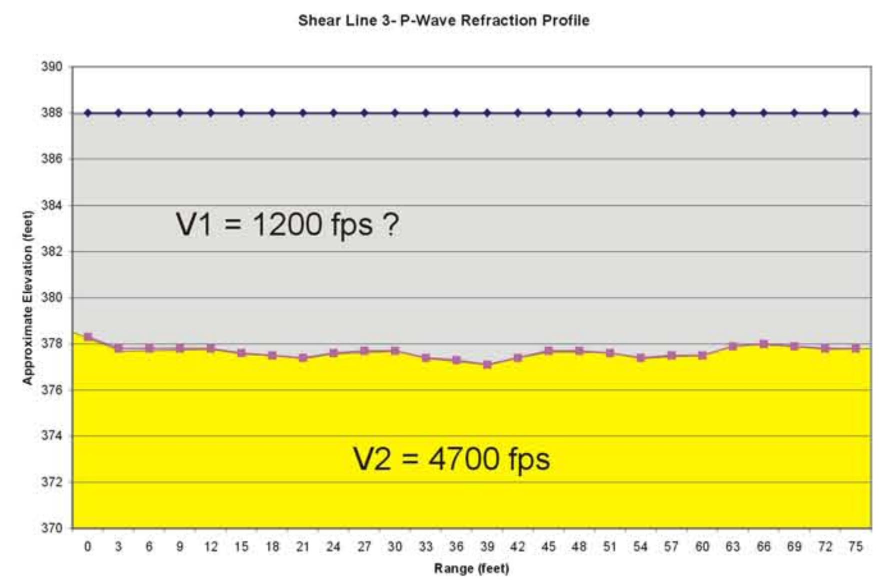
Sw-1 Refraction Profile



Sw-2 Refraction Profile



Sw-3 Refraction Profile



Note: Surface elevations were obtained from a USGS Topographic map of the area obtained through TerraServer-USA.com

Refraction Profiles Along Shear Wave Lines
Genesis Project Shear Wave Investigation
Riverside County, California

SCALE: See Diagrams			DRAWN BY: J.J.R.
DATE: 9-17-2009	JOB NUMBER:	129-263-09	REVISED:

J R ASSOCIATES Civil and Environmental Geophysics
1886 Emory Street, San Jose, CA (408) 293-7390

DRAWING NUMBER: 6

PRELMINARY GEOTECHNICAL AND GEOLOGIC HAZARDS INVESTIGATION FOR GENESIS SOLAR ENERGY PROJECT CHUCKWALLA VALLEY RIVERSIDE COUNTY, CALIFORNIA

October 2009

Prepared for

WorleyParsons
2330 East Bidwell, Suite 150
Folsom, California 95630

Project No. 2341-1

ROMIG ENGINEERS, INC.
GEOTECHNICAL & ENVIRONMENTAL SERVICES

October 20, 2009
2341-1

WorleyParsons
2330 East Bidwell, Suite 150
Folsom, California 95630

RE: PRELIMINARY GEOTECHNICAL AND GEOLOGIC HAZARDS INVESTIGATION GENESIS SOLAR ENERGY PROJECT RIVERSIDE COUNTY, CALIFORNIA

Gentlemen:

In accordance with your request, we have performed a preliminary geotechnical and geologic hazards investigation for the proposed Genesis Solar Energy Project. The Project site is located in the Chuckwalla Valley in Riverside County about 25 miles west of Blythe, California.

The accompanying report summarizes the results of the field exploration conducted in the Project area, laboratory testing, and geologic and engineering analysis, and presents our preliminary geologic and geotechnical recommendations for the proposed energy facilities. These preliminary recommendations are based on limited field exploration and laboratory testing and will be updated after on-site exploratory borings are advanced during the design-level geotechnical investigation. The findings from our preliminary investigation indicate the site is feasible for the proposed development provided the recommendations presented in this report and in the design-level geotechnical report are incorporated into project design and construction. We refer you to the text of our report for specific recommendations.

Thank you for the opportunity to work with you on this Project. Please call if you have any questions or comments concerning the findings or recommendations from our preliminary investigation.

Very truly yours,

ROMIG ENGINEERS, INC.

REGISTERED PROFESSIONAL ENGINEER GLENN A. ROMIG 002157 EXP. 12-31-09 GEOTECHNICAL STATE OF CALIFORNIA

Michael Tietze

REGISTERED GEOLOGIST MIKE TIETZE No. EG1803 Exp. CERTIFIED ENGINEERING GEOLOGIST STATE OF CALIFORNIA

Glenn A. Romig, P.E., G.E.

Michael Tietze, P.G, C.E.G., H.G.

Copies: Addressee (6)

GAR:MT:CMS:jh

PRELIMINARY GEOTECHNICAL AND GEOLOGIC HAZARDS INVESTIGATION
GENESIS SOLAR ENERGY PROJECT
CHUCKWALLA VALLEY
RIVERSIDE COUNTY, CALIFORNIA

PREPARED FOR:
WORLEYPARSONS
2330 EAST BIDWELL, SUITE 150
FOLSOM, CALIFORNIA 95630

PREPARED BY:
ROMIG ENGINEERS, INC.
1390 EL CAMINO REAL, SECOND FLOOR
SAN CARLOS, CALIFORNIA 94070

OCTOBER 2009

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PRELIMINARY GEOTECHNICAL AND GEOLOGIC HAZARDS INVESTIGATION FOR GENESIS SOLAR ENERGY PROJECT CHUCKWALLA VALLEY RIVERSIDE COUNTY, CALIFORNIA

INTRODUCTION

This report presents the results of our preliminary geotechnical and geologic hazards investigation for the proposed Genesis Solar Energy Project. The Project site is located in the northeastern portion of the Chuckwalla Valley in Riverside County, California between the communities of Blythe (approximately 25 miles to the east) and Desert Center (approximately 27 miles to the west). The location of the Project site is shown on the Vicinity Map, Figure 1.

The purpose of this study was to provide preliminary geotechnical and geologic recommendations for the solar energy project based on available field and laboratory data and is intended to supplement information provided in Section 5.5 Geologic Resources and Hazards in the Genesis Solar Energy Project, Application for Certificate, submitted by Genesis Solar, LLC. to the California Energy Commission on August 31, 2009. Further geotechnical investigation including on-site exploratory borings will be required for the energy facilities and infrastructure to prepare design-level recommendations.

Project Description

The Genesis Solar Energy Project (the Project) will consist of two independent concentrated solar electric generating facilities with a nominal net electrical output of 125 megawatts (MW) each, for a total net electrical output of 250MW. The Project will generate heat used for stream production and power generation with a stream turbine generator. Groundwater will be used as the water supply for cooling, stream cycle make-up water, mirror washing, and potable water supply. Two power blocks containing the steam turbine and power generating facilities will be located centrally surrounded with large areas of relatively lightly loaded parabolic trough solar collectors. The power blocks and solar arrays will occupy about 1360 acres. Additionally, evaporation ponds, detention basins, the linear corridor (includes access road, transmission lines, natural gas line), administration buildings, other support facilities, bioremediation land treatment areas, and some open areas increase the total Project area to approximately 1,826 acres. The general layout of the Project is shown on the Conceptual Grading Plan, Figure 2

The Project includes relatively heavy facilities in the power block areas, including steam turbine/generators (STG) and condensers, solar steam generators (SSG), cooling towers, natural gas-fired auxiliary boilers, heat exchangers, and other ancillary equipment and tanks. Some of the power generation facilities at the power blocks are expected to have relatively high structural loads while the solar collectors are relatively light.

Each solar collector array will be supported by structures (stands) that connect the parabolic troughs to a drive mechanism. Each array will be supported by multiple individual foundations with a foundation located approximately every 40 feet along the array.

The Project will include a common administration building and warehouse between the two, 125 MW power plants, a control building in each power block, a water treatment building, as well as a number of pre-engineered enclosures for mechanical and electrical equipment. The total square footage of the various Project buildings and pre-engineered enclosures (e.g., control rooms, administration building, warehouse, electrical equipment enclosures, fire pumps, and diesel generators) is approximately 39,000 square feet.

There will be a number of covered water tanks on site for each 125 MW power plant including a 500,000-gallon raw water storage tank, a 1,250,000-gallon treated water storage tank, a 250,000 waste water storage tank, and a 40,000-gallon storage tank for storage of demineralized water. Water storage tanks will be vertical, cylindrical, field-erected steel tanks supported on foundations consisting of either a reinforced concrete mat or a reinforced concrete ring beam.

Only a small portion of the overall plant site will be paved, primarily the site access road and portions of each power block (paved parking lot and roads encircling the STG and SSG areas). The remaining portions of the power block will be gravel surfaced. The solar field will remain unpaved and without a gravel surface in order to prevent rock damage from mirror wash vehicle traffic; an approved dust suppression coating will be used on the dirt roadways within and around the solar field.

Grading for post-developed conditions will slightly modify the existing contours to provide a level surface required for the parabolic troughs and graded pads for the power blocks. Grading will also be required for the evaporation pond and retention pond excavations, the protective berms and drainage channels along the northern upslope side of the Project site, drainage channels, access roads and other improvements.

Linear elements of the Project include transmission lines, an access road, and a gas pipeline. The 6.5-mile-long access road will extend southeast to the Wiley's Well Road exit of Interstate Highway 10. The transmission lines and gas pipelines will parallel the access road alignment. The transmission lines will continue south beyond Interstate Highway 10 where they will share the transmission poles of the Blythe Energy Transmission Line and eventually connect to the Southern California Edison (SoCal Edison) Colorado River substation. The alignment of the off-site linears is shown on the Area Geologic Map for Off-Site Linears, Figure 3.

Scope of Work

The scope of our work for this investigation was presented in our agreement with WorleyParsons dated June 1, 2009. In order to accomplish this investigation, we performed the following work.

- Review of literature in our files and available information regarding geologic, geotechnical, and seismic hazards in the vicinity of the Project site.

- Site reconnaissance and surface soil collection by our staff geologist.

- Review of soil samples and the exploration log prepared by WorleyParsons from the Well Test Boring OBS-2 located approximately 1.5 miles west of the Project site.

- Laboratory testing of selected samples from Well Test Boring OBS-2 to aid in soil classification and to help evaluate the engineering properties of the soils encountered. Laboratory testing included moisture content, grain size, and plasticity. In addition, corrosion potential tests were performed on two samples of surface soil collected during our site reconnaissance.

- Review of shear wave velocity data collected across the Project site to help assess subsurface conditions.

- Geologic and geotechnical analysis and evaluation of the resulting subsurface and laboratory data to develop preliminary geologic and geotechnical design criteria.

- Preparation of this report presenting our preliminary geologic and geotechnical findings and recommendations for the Project.

Limitations

This report has been prepared for the exclusive use of WorleyParsons for specific application to developing preliminary geotechnical and geologic design criteria for the Genesis Solar Energy Project to be constructed in the Chuckwalla Valley in Riverside County, California. We make no warranty, expressed or implied, except that our services were performed in accordance with geologic and geotechnical engineering principles generally accepted at this time and location. This report was prepared to provide engineering opinions and preliminary recommendations only based upon the limited subsurface data available at this time. These conclusions and recommendations presented in this report should be considered preliminary until the subsurface conditions are adequately evaluated with exploratory borings at the site of the improvements.

The analysis, conclusions, and preliminary recommendations presented in this report are based on site conditions as they existed at the time of our study; the currently planned solar power plants; review of readily available reports and boring data relevant to the site conditions; and laboratory test results. In addition, it should be recognized that certain limitations are inherent in the evaluation of subsurface conditions, and that certain conditions may not be detected during an investigation of this type. Changes in the information or data gained from any of these sources could result in changes in our conclusions or recommendations. If such changes occur, we should be advised so that we can review our report in light of those changes.

SITE RECONNAISSANCE AND SUBSURFACE EXPLORATION

Site reconnaissance and collection of surface soil samples was performed by our staff geologist on July 30, 2009. Available subsurface exploration information provided to us for the Project site included the log of test well boring OBS-2 drilled under the direction of WorleyParsons from May 28 to July 2, 2009 and the results of geophysical testing performed by JR Associates. The location of the test well boring is shown on the Site Geologic Map, Figure 7. A log of the upper 75 feet of the test well boring and the results of laboratory tests we performed on samples of soil WorleyParsons provided to us from the test well boring is attached in Appendix A. The geophysical survey is attached in Appendix B.

Surface Conditions

The Project site lies on a broad, relatively flat, very gently south-sloping surface covered with alluvial deposits. The alluvial deposits, derived from the surrounding mountains, have formed fans that coalesce into a single bajada surface that wraps around the mountain fronts. Between the bajada surfaces from each mountain chain lies a broad valley-axial drainage that extends southward between the mountains and drains to the Ford Dry Lake playa, located about 1 mile south of the site. The Project site generally slopes from north to south with elevations of approximately 400 to 370 feet above mean sea level.

The majority of the eastern part of the Project site is characterized by subdued bar and swale topography at ground level and lacks water erosional features. Very few small washes are continuous across the eastern part of the Project site.

WorleyParsons Well Test Boring

WorleyParsons advanced Well Test Boring OBS-2 using dual tube reverse circulation drilling equipment. The boring was advanced to a depth of 900 feet below the ground surface (bgs). Soil samples were obtained at 5 to 10 foot depth intervals by a member of WorleyParsons staff and were classified with additional laboratory testing from the ground surface to a depth of 75 feet. Although the test well boring was located about 1.5 miles west of the Project site, it provides useful information regarding density, plasticity, and type of soil present in the geologic environment of Project site area.

The soils encountered at the well test boring consisted of interbedded silty and gravelly sands and sandy lean clays to sandy fat clays that are alluvial and possibly lacustrine in nature. The depth to first encountered ground water at the well test boring was estimated at 77 feet bgs.

Two samples of clay obtained from the boring had Liquid Limits of 39 and 58 and Plasticity Indices of 23 and 39. These test results indicate the clay strata encountered in the boring on the Project site have moderate to high plasticity. Four sand samples obtained from the boring were washed through an ASTM No. 200 Sieve with 16 percent to 48 percent passing. Free swell tests performed on three samples of clay indicated free swells ranging from 130 to 270 percent. The plasticity and free swell test results suggest the clays encountered in the boring have a variable potential for expansion and are locally very plastic and potentially moderately to highly expansive where present near surface improvements.

Based on our site observations and laboratory classification of the soil samples from the well test boring, the near surface alluvial soils in the area of the Project site are expected to consist of a veneer of primarily granular soil from 1 to several feet thick underlain by materials that vary locally in soil type, degree of plasticity, and potential for expansion.

We refer you to the boring log and test results presented in Appendix A for detailed descriptions of the soils encountered in the boring and the results of our laboratory testing.

Geophysical Testing

Two geophysical surveys were performed at the Project site. The initial study (JR Associates, August 26, 2009) included seismic refraction and electromagnetic soundings at several locations across the Project site. Based on this data, general subsurface conditions appeared to be relatively uniform laterally across the Project site. Groundwater was estimated to be brackish and vary in depth from about 61 to 81 feet below the ground surface. This testing suggested the subsurface alluvium at the Project site is rich in clay.

The second study (JR Associates, September 21, 2009) measured shear wave velocity profiles using the multichannel analysis of surface wave method (MASW) at three locations across the Project site and an additional location at the well test boring. This survey is attached in Appendix B. The shear wave data indicated a surfical layer about 10 feet thick with average velocity of about 950 feet per second (fps); underlain by what was interpreted to be a 15 foot thick weakly cemented layer with average velocity of about 1700 fps; underlain by a 25 foot thick relatively softer strata with average velocity of about 730 feet per second fps; in turn underlain between 45 and 75 feet by denser alluvium with average shear wave velocity of about 1370 fps. The average shear wave velocity at these three locations over the upper 75 feet was estimated at about 1200 fps, which was nearly identical (within 3 percent) with the average MASW velocity at the test well boring site.

Downhole shear wave velocities were also determined at the test well boring location. The average shear wave velocity in the upper 75 feet was estimated at about 1050 fps. The report concluded that the MASW profiles may overestimate the average velocity by about 13 percent at the Project site.

GEOLOGIC AND SEISMIC SETTING

Regional Geology and Physiography

The Project site and off-site linears are situated within northeastern portion of Chuckwalla Valley, an east-southeast trending valley in California's Mojave Desert Geomorphic Province. The Mojave Desert Geomorphic Province is a wedge-shaped interior region separated from the Sierra Nevada and Basin and Range Provinces to the northwest by the Garlock Fault and its eastward extensions, and is bounded to the southwest by the Transverse Range and Colorado Desert Provinces, the San Andreas Fault, and its southern extensions. The Mojave Desert Geomorphic Province is characterized by northwest-southeast as well as east-west trending structures and mountain ranges, separated by desert valleys and plains with many enclosed drainages and playas.

The Chuckwalla Valley is bounded by the Chuckwalla, Little Chuckwalla, and Mule mountains on the south, the Eagle Mountains on the west, the Mule and McCoy mountains on the east, and the Coxcomb, Granite, Palen, and Little Maria mountains on the north. The elevation of Chuckwalla Valley ranges from under 400 feet at Ford Dry Lake, just south of the Project site, to approximately 1,800 feet above mean sea level (amsl) west of Desert Center and along the upper portions of the alluvial fans that ring the valley flanks. The surrounding mountains rise to approximately 3,000 and 5,000 feet amsl.

The region has undergone a complex geologic history that includes volcanic activity, faulting, folding, uplift, erosion, and sedimentation. The Project area is underlain by Holocene to Miocene basin fill deposits (Stone, 2006). These deposits include younger alluvium, older (Pleistocene) alluvium, the Pliocene Bouse Formation, and the Miocene fanglomerate. The uppermost alluvium in the basin consists of Holocene to Pleistocene alluvial fan, valley axial (fluvial), playa (dry lake), and aeolian (wind blown) deposits. The geology of the Project area and the off-site linears is shown on the Area Geologic Maps on Figures 3 and 4.

Regional Tectonic Setting

The Mojave Desert comprises an area bounded by the seismically active Salton Trough to the west and southwest, and the Garlock Fault to the north. To the east and southeast it is bounded by the Sonoran Desert subprovince, a relatively stable tectonic region located in southeastern California, southwestern Arizona, southern Nevada, and northern Mexico (Balderman, et al., 1978). Chuckwalla Valley is located in the eastern Mojave Desert province in an area that is relatively stable tectonically. Faults in the area occur primarily in Tertiary and pre-Tertiary strata and are related to compressional tectonism along a convergent Andean and Island arc margin in the Mesozoic, and extensional detachment and block faulting during Tertiary time. As shown on the Regional Fault Location Map, Figure 5, no faults of Quaternary age are known to exist near the Project site.

Engineering Geologic Reconnaissance and Site Geology

Our engineering geologic reconnaissance was conducted on July 30 and September 23, 2009 and consisted of walking the Project site to observe the topography and surface conditions. The surface was generally covered by a veneer of predominantly granular soil exhibiting subdued bar and swale topography at ground level with few very small washes and few water erosional features. Lag deposits and small vegetated mounds were observed. Subsurface stratification and cross bedding was observed in the top 12 to 18 inches suggesting migrating ripples and formation of silt crusts after sheet floods. Two deeper soil pits encountered dense soils and buried soil horizons with some carbonate deposition at depths of about 2 feet beneath an active surface alluvial layer without soil development. West of the Project Site boundaries, relict soils with carbonate horizon development were observed to locally protrude through more recent alluvial deposits at the edges of washes. This suggests that the site is underlain by a thin veneer of recent alluvial material deposited by sheet floods overlying older alluvium with some soil horizon development.

Regional Seismicity

The Project site and off-site linears lie within the Sonoran Desert subprovince, which is a relatively stable tectonic region located in southeastern California, southwestern Arizona, southern Nevada, and northern Mexico. Review of the California Department of Conservation's Map Sheet 49, Epicenters and Areas Damaged by M>5 California Earthquakes 1800 – 1999, indicates that eastern Riverside County did not experience any damaging earthquakes or ground shaking during this period (Toppozada and others, 2000). The locations of Quaternary and younger faults and historical earthquake epicenters within 100 kilometers of the Project site are shown on Figure 6 and indicate that more seismically active areas are located to the west, southwest, and northwest of the area. As shown on these figures and discussed further below under the section titled "Local Faulting and Seismicity," the nearest fault defined by the State of California as "Sufficiently Active" is located more than 46 miles (74 km) from the Project site.

Local Geology

The Project site has been mapped as being underlain by Holocene to Pleistocene age Quaternary Alluvial deposits consisting of alluvial fan, valley axial, lacustrine, and playa deposits. These deposits generally consist of fine gravel, fine to coarse sand, silt, and clay (DWR, 1963). The Pliocene Bouse Formation underlies the Quaternary sediments. The unit generally consists of a basal limestone overlain by interbedded clay, silt, sand, and tufa that may include lacustrine sediments. The Bouse Formation is unconformably underlain by a fanglomerate of Miocene to Pliocene age, which consists of angular to subrounded and poorly sorted, partially to fully cemented pebbles with a sandy matrix (Metzger and others, 1973). Bedrock beneath the Project site consists of metamorphic and igneous intrusive rocks of pre-Tertiary age that form the basement complex (DWR, 1963).

Three lines of evidence have been used to describe and confirm the geologic conditions underlying the Project site. First, geophysical investigations conducted at the Project site indicate that the electrical conductivity of the underlying sediments (an indicator of the amount of fine grained sediment and salinity of the groundwater) is consistent and similar across the Project site area. Second, seismic refraction profiling suggests that the shallow alluvium has similar properties across the Project site. Third, subsurface investigation at the well site demonstrated the Project area is underlain by alluvium consisting of interbedded and intermixed dense sand and gravel, and hard silt and clay to a depth of approximately 245 to 275 feet bgs. These sediments are heterogeneous both laterally and vertically, although the valley axial alluvium beneath the eastern portion of the Project site may contain cleaner sands than sediments underlying the bajada surfaces, and laterally may be more homogenous.

Beneath the alluvium, the Pliocene Bouse Formation is estimated to extend to approximately 2,000 bgs and is generally richer in fine grained sediments than the overlying alluvium. The Miocene fanglomerate is inferred to underlie the alluvium at this depth and is estimated to extend to approximately 2,900 feet bgs. The geology of the Project site and off-site linears is shown on the Site Geologic Maps, Figures 7 and 8.

Local Faulting and Seismicity

The Project site and off-site linears lie within the eastern part of Riverside County in a part of California considered not to be very seismically active. Although there are several bedrock faults off-site in the mountains surrounding Chuckwalla Valley, these do not exhibit recent activity and are presumed to be Tertiary or pre-Tertiary in age (Stone, 2006). In addition, gravity anomalies suggest the presence of several subsurface faults beneath Chuckwalla Valley in the vicinity of the Project area (Stone, 2006; Rotstein, et al., 1976). The gravity anomalies reflect abrupt changes in basement elevation strongly suggestive of dip-slip movements. These faults are presumed Tertiary and likely inactive with a very low chance of producing earthquakes.

The active faults considered most likely to produce large earthquakes potentially affecting the Project site are located at a considerable distance to the west and southwest, and include the San Andreas, Imperial, and San Jacinto-Anza faults. Thus, the likelihood of surface rupture occurring from active faulting at the Project site is remote. Other smaller faults are located within approximately 100 kilometers (km) of the Project site as summarized below. These faults are believed to be capable of producing ground shaking with peak ground accelerations exceeding 0.10 times the force of gravity (0.10 g).

Table 1. Sufficiently Active Faults within 100 Kilometers of the Project site

Fault Name	Approximate Distance and Direction from Project site	Slip Rate (mm/year)	Maximum Earthquake Magnitude
San Andreas Fault	46 miles (74 km) southwest	>5	7.4
Brawley Seismic Zone	47 miles (76 km) southwest	1 to >5	7.2
Pinto Mountain Fault	54 miles (86 km) west-northwest	1 to 5	7.0
Pisgah-Bullion Fault	57 miles (91 km) northwest	0.2 to 1	7.1
Imperial Fault	61 miles (98 km) southwest	>5	7.0
San Jacinto-Anza Fault	61 miles (98 km) southwest	1 to >5	7.2

Fault locations and slip rates taken from USGS Earthquake Hazard Program Quaternary Fault and Fold Database (http://gldims.cr.usgs.gov/qfault/viewer.htm), using latitude 33.67 degrees, longitude -115.00 degrees as the Site coordinates.

Maximum Magnitude and Slip Rate taken from California Department of Conservation, Division of Mines and Geology, 1998, Maps of Known Active Fault Near-Source Zones in California and Adjacent Portions of Nevada.

A preliminary estimate of ground motions expected at the Project site was prepared using source and attenuation models developed by the USGS National Seismic Hazard Mapping Project (NSHMP, 2009). For design of important facility structures, a site-specific Probabilistic Seismic Hazard Assessment will be completed as part of a design-level Geotechnical Investigation and will be made available to the CEC. The preliminary results indicate that the peak ground acceleration (PGA) with a probability of exceedance of 10 percent in 50 years (475 Year Return Period) is 0.15 units of gravity (g). The deaggregation information indicates the mean moment magnitude is 6.8 at a mean distance of 68 km. The PGA with a probability of exceedance of 2 percent in 50 years (2475 Year Return Period) is 0.24 g. The mean moment magnitude is 6.7 at a mean distance of 48 km. Figures 9 and 10 show the deaggregation data for the 475- and 2475-year return periods.

Table 2 below presents seismic design parameters based on the 2007 California Building Code (CBC). These seismic design parameters may be used for design of structures where appropriate. On the basis of available data regarding on-site geologic conditions and information from subsurface exploration at the well site located east of the power plant sites and the recent geophysical testing, the Project site and off-site linears may be classified as Site Class D, Stiff Soil Profile. The site class and parameters listed below will be updated as appropriate based upon the results of the design-level geotechnical investigation.

Table 2. 2007 CBC Seismic Design Parameters

Parameter	**Value**
Site Class	D
Ss- Mapped Spectral Acceleration, Short Period	0.478 g
S1- Mapped Spectral Acceleration, Long Period	0.249 g
Fa-Site Coefficient, Short Period	1.417
Fv-Site Coefficient, Long Period	1.901
SDs-Design Spectral Response Acceleration, Short Period	0.452 g
SD1-Design Spectral Response Acceleration, Long Period	0.316 g
SMs-MCE* Spectral Response Acceleration, Short Period	0.678 g
SM1-MCE* Spectral Response Acceleration, Short Period	0.474 g

Calculated using USGS Program “Earthquake Ground Motion Parameters” Version 5.0.9 based on latitude 33.67 degrees and longitude -115.00 degrees as the site coordinates.

MCE = Maximum Considered Earthquake

SEISMIC AND GEOLOGIC HAZARDS

As part of our investigation, we briefly reviewed the potential for geologic hazards to impact the site considering the geologic setting and the soils encountered during this preliminary investigation. Because there are no open bodies of water in the vicinity of the Project site or the off-site linears, tsunami and seiche hazards do not exist for the Project. The results of our review are presented below.

- Seismic Ground Shaking - Although the Project site is not located in a very seismically active area, it may be subjected to ground shaking from movement along one or more of the sufficiently active or well-defined faults in the adjacent area. The Riverside County General Plan, Safety Element (Riverside County, 2008) indicates that the Project site and the off-site linears associated with the Project are in an area of moderate ground shaking risk, where peak ground accelerations may reach 0.1 to 0.2 g. Our preliminary seismic hazard analysis indicates the peak ground acceleration with a probability of exceedance of 10 percent in 50 years is 0.15 g. In our opinion, the Project site and the off-site linears are subject to low to moderate seismic ground shaking hazard.

- Ground Rupture - The Project site is not located within a State of California Earthquake Fault Zone designated by the Alquist-Priolo Special Studies Zone Act of 1972 (formerly known as a Special Studies Zone), an area where the potential for fault rupture is considered probably (Riverside County, 2008). In addition, no Quaternary, Sufficiently Active, or Well Defined Faults are located under or near the Project site. Based on this information and engineering judgment, earthquake-induced ground rupture is not considered to be a significant hazard at the Project site and the off-site linears associated with the Project.

- Slope Stability - The Project site and off-site linears associated with the Project are not considered to be in an area with the potential for permanent ground displacement due to static or earthquake-induced landslides because surface topography at and near the Project site is relatively flat (Riverside County, 2008). A review of the Riverside County General Plan, Safety Element, did indicate areas considered susceptible to earthquake induced landslides and rockfalls in the Palen and McCoy Mountains; however, these areas are several miles from the Project site and are not expected to impact the Project. Based on this information and engineering judgment, slope instability is not considered to be a significant hazard at the Project site and the off-site linears associated with the Project.

- Erosion - Erosion is the displacement of solids (soil, mud, rock, and other particles) by wind, water, or ice, and by downward or down-slope movement in response to gravity. Due to generally flat terrain, the Project site is not prone to significant mass wasting (gravity-driven erosion and non-fluvial sediment transport) at present. The Riverside County General Plan, Safety Element (Riverside County, 2008), indicates the Project site and the off-site linears associated with the Project are in an area with moderate potential for wind erosion, the off-site linears are in areas with moderate to high potential for wind erosion. Soil characteristics at the Project site and off-site linears allow for the potential for wind and water erosion, and significant sediment transport currently occurs across the valley axial drainage that crosses the majority of the proposed plant site. As indicated above, these deposits are characterized by subdued bar and swale topography and on-going alluvial transport from sheet floods. Limited sand and aeolian erosion also occurs between depositional episodes.

  Soil erosion from wind and water during construction activities is probable. Under current conditions, the soil loss is estimated to be about one ton per year from the Project site and areas of off-site linears associated with the Project. Construction activities without implementation of Best Management Practices (BMPs) would result in a potential for soil loss of about 1,400 tons. The implementation of BMPs is expected to reduce water and wind erosion of soils during construction to less than significant levels.

To address the management of sediment transport, erosion, and sedimentation during operation, the Project design will incorporate diversion berms, channels, detention basins, and dispersion structures. The final design of these features will be developed during detailed design, and will include industry-standard calculations and modeling to reduce the potential for erosion or sedimentation, and to reduce the need for on-going maintenance. Dirt roads and exposed surfaces will be periodically treated with dust palliatives as needed to reduce wind erosion. Construction and maintenance of the proposed drainage and sediment management system at the Project site is expected to reduce water and wind erosion at and downstream of the Project site to less than significant levels.

- Liquefaction - Liquefaction is a condition that occurs when seismically-induced ground motions cause soil densification resulting in an increase in soil pore water pressure in saturated soils resulting in loss of soil shear strength. The soils most prone to liquefaction are loose to medium dense, uniformly-graded sands, silty sands, and sandy silts. The effects of soil liquefaction can include loss of bearing strength, differential settlement, ground oscillations, lateral spreading, and flow failures or slumping. Liquefaction occurs primarily in areas where groundwater is less than 50 feet below the ground surface. The Riverside County General Plan Safety Element (Riverside County, 2008) indicates that the majority of Chuckwalla Valley, including the soils beneath the Project site and associated Project off-site linears, is mapped as having deep groundwater but underlain by soils with an otherwise moderate susceptibility to liquefaction. The depth to water beneath the Project site is estimated to range from approximately 61 to 94 feet bgs. In addition, the sandy soils encountered in the upper 100 feet below the ground surface in the well test boring for this preliminary study were generally dense to very dense and well-graded. Dense, well-graded sands are not generally considered susceptible to liquefaction. Based on this information and engineering judgment, the potential for soil liquefaction at the Project site and along the associated Project off-site linears is considered to be low. This will be confirmed during the design-level geotechnical investigation.

- Subsidence - Subsidence, or a lowering of surface elevation due to removal of subsurface support, can result from several causes and ranges from small or local collapses to broad regional lowering of the earth's surface. Potential causes of subsidence include tectonic movement, seismic compaction, hydrocompaction, consolidation induced by groundwater withdrawal, and consolidation under applied loads. Of greatest concern to structures at the Project site is localized or differential settlement that can damage foundations, structures, and surface improvements at the Project site. More widespread subsidence has regional implications and can be damaging to regional drainage, water conveyance, flood control, and other factors.

Ground subsidence can occur as a result of water level decline in aquifer systems. When the water pressure in an aquifer is reduced as a result of lowering of the groundwater level, the resultant increase in effective stress causes the “skeleton” of the aquifer system to deform slightly. Reversible deformation occurs in all aquifer systems as a result of the cyclical rise and fall of groundwater levels associated with short and longer term climatic cycles. Permanent ground subsidence can occur when the groundwater level in the aquifer falls below its lowest historical level, and the particles in the aquifer skeleton are permanently rearranged and compressed. Soils particularly susceptible to such consolidation and subsidence include compressible clays. This type of deformation is most prevalent when confined alluvial aquifer systems are overdrafted, resulting in water level declines of tens or hundreds of feet.

Based on the general geology of the Chuckwalla Valley, the Riverside County General Plan, Safety Element designates basin fill sediments in the valley as being susceptible to subsidence (Riverside County, 2008). However, subsidence has not been reported in the valley. Groundwater demand in the valley was at a maximum in the 1980s and 1990s, when agricultural pumping was estimated to exceed 48,000 acre-feet per year. Current agricultural groundwater demand is estimated to be less than 2,000 acre-feet per year, and with implementation of the proposed Project (water demand of approximately 1,600 acre-feet per year) the cumulative water demand in the basin is anticipated to remain well below the historical maximum. As such, it is not likely that water levels in the Bouse Formation aquifer will drop below their historical low levels. In addition, the clays encountered during drilling of the boring for the test well program at the Project site were hard and highly over-consolidated. Based on this information and engineering judgment, the potential for significant subsidence associated with pumping of groundwater for the Project is considered low.

Seismically-induced settlement can occur as a result of moderate and large earthquakes due to compression of soft or loose, natural or fill soils located above the groundwater table. This seismically-induced settlement can cause damage to surface and near-surface structures. The soils most susceptible to seismic settlement are clean, loose, granular soils. Due to the expected dense to very dense nature of the surface and near-surface soils, the potential for damage due to seismically-induced settlement is considered to be low at the Project site and associated Project off-site linears.

- Collapsible Soil Conditions - Alluvial soils in arid and semi-arid environments can have characteristics that make them prone to collapse with increase in moisture content and without increase in external loads. Soils that are especially susceptible to collapse or hydrocompaction in a desert environment are loose, dry, sands and silts, and soils that contain a significant fraction of water soluble salts. In the Project site vicinity, this would include aeolian sand, playa evaporite deposits, and loose flash flood deposits. Based on surface reconnaissance, review of geologic mapping, and review of aerial photographs, there are aeolian deposits south of the Project site near Ford Dry Lake, but no significant aeolian or playa deposits are located within the Project site. There do not appear to be near-surface evaporite deposits associated with Ford Dry Lake (Stone, 2006). The near-surface soils at the Project site and associated Project off-site linears are composed primarily of alluvial soils that appear to have been deposited in relatively thin sheet flood and fluvial deposits that have a low potential for hydrocompaction. Based on this data and engineering judgment, the Project site soils do not have a significant potential for hydrocompaction or collapse. Some areas along the off-site linears are underlain by significant aeolian deposits that may have low to moderate potential for hydrocompaction. This will be further evaluated and addressed during the design-level geotechnical investigation.

- Expansive Soil - Expansive soil is predominantly fine-grained and contains clay minerals capable of absorbing water into their crystal structure. Expansive soil is often found in areas that were historically a flood plain or lake area, but can also be associated with some types of shale, volcanic ash, or other deposits, and can occur in hillside areas also. Expansive soil is subject to swelling and shrinkage, varying in proportion to the amount of moisture present in the soil. As water is initially introduced into the soil (by rainfall or watering) expansion takes place. If dried out, the soil will contract, often leaving small fissures or cracks. Excessive drying and wetting of the soil can progressively deteriorate structures that are not designed to resist this effect, and can lead to differential settlement of buildings and other improvements. The uppermost surface soils consist of predominately granular soil, but near-surface soils at the Project site and off-site linears may have clayey interbeds with a moderate to high expansion potential. If expansive soils are identified near the ground surface during the design-level geotechnical investigation, recommendations will be provided to mitigate the effects of the expansive soils on pavements, structures, and concrete slabs-on-grade.

CONCLUSIONS AND RECOMENDATIONS

From a geologic and geotechnical viewpoint, the Project site and associated off-site linears are suitable for the proposed development, provided the recommendations presented in this preliminary report and in the design-level geotechnical report are followed during design and construction. Based on our site reconnaissance, the soils encountered at the well site, the results of the geophysical survey, and the geologic environment of the Project site, unusually weak or compressible soils are not expected to be a concern. There is a potential for expansive soil interbeds to be present below portions of the Project site or off-site linears and for shallow, loose aeolian sands that could be susceptible to hydrocompaction in the area of the off-site linears. The extent of these soils will be identified during the design-level geotechnical investigation and addressed during project design.

The following sections of this report present preliminary earthwork and foundation design recommendations for the Project. These recommendations will be updated after on-site exploratory borings and addition laboratory testing and office evaluation are performed during the design-level geotechnical investigation.

EARTHWORK

General

Grading for the Project is expected to include but not be limited to earthwork to create level building pad areas for each of the power blocks, the administration building/ warehouse, leveling of the solar fields as needed, excavations for the evaporation ponds and detention basins and the construction of diversion berms. Paved and unpaved roads and utility trenches will also be constructed as well as a drainage berm along the north side of the Project area and drainage channels in selected areas of the Project site.

Remedial grading may be required below sensitive structures or pavements where locally loose, compressible, or expansive soils are encountered during the design-level geotechnical investigation. The extent of the required remedial grading will be established during the design-level geotechnical investigation. In general, loose surface soils and/or expansive soils should be removed to at least 24-inches below the slab, foundation, or pavement section and replaced with non-expansive select structural fill.

Clearing and Subgrade Preparation

All deleterious materials, such as vegetation, root systems, etc., should be cleared from areas of the site to be built on, paved, or otherwise developed. Excavations that extend below finished grade should be backfilled with structural fill that is water-conditioned, placed, and compacted as recommended in the section of this report titled “Compaction.”

After the site has been properly cleared, stripped, and excavated to the required grades, exposed soil surfaces in areas to receive structural fill, structures, concrete slabs-on-grade or pavements should be scarified to a depth of 6 inches, moisture conditioned, and compacted as recommended for structural fill in the section of this report titled "Compaction."

Material for Fill

All on-site soil containing less than 3 percent organic material by weight (ASTM D2974) may be suitable for use as structural fill. Structural fill should not contain rocks or pieces larger than 6 inches in greatest dimension and no more than 15 percent larger than 2.5 inches. Imported soil and non-expansive fill should have a Plasticity Index no greater than 12, should be predominately granular, and should have sufficient binder so as not to slough or cave into foundation excavations or utility trenches. All proposed import material should be evaluated by a member of our staff prior to delivery to the site.

Temporary Slopes and Excavations

The contractor should be responsible for the design and construction of all temporary slopes and any required shoring. Shoring and bracing should be provided in accordance with all applicable local, California, and federal safety regulations, including current OSHA excavation and trench safety standards.

Because of the potential for variation of the on-site soils, field modification of temporary cut slopes may be required. Unstable materials encountered on slopes during and after excavation should be trimmed off even if this requires cutting the slopes back to a flatter inclination. Protection of structures near excavations and trenches will also be the responsibility of the contractor.

Finished Slopes

Finished slopes be cut or filled to an inclination no steeper than 2:1 (horizontal:vertical). Finished slopes for the evaporation ponds, drainage channels, and diversion berms should not exceed a gradient of 3:1. Exposed slopes may be subject to minor erosion and sloughing that would require periodic maintenance.

Compaction

Scarified surface soils and all structural fill should be compacted in uniform lifts no thicker than 8-inches in uncompacted thickness, conditioned to the appropriate moisture content, and compacted as recommended for structural fill in Table 3 below. The relative compaction and moisture content recommended in Table 3 is relative to ASTM Test D1557, latest edition.

Table 3. Compaction Recommendations

	Relative Compaction*	Moisture Content*
General		
• Scarified subgrade in areas to receive structural fill.	90 percent	At least 2 percent above optimum
• Structural fill composed of native soil or non-expansive fill.	90 percent	At least 2 percent above optimum
• Structural fill composed of highly expansive soil.	87 to 92 percent	At least 3 percent above optimum
Pavement Areas		
• Upper 8-inches of soil below aggregate base.	95 percent	Above optimum
• Aggregate base.	95 percent	Near optimum
Utility Trench Backfill		
• On-site soil.	90 percent	At least 2 percent above optimum
• Imported sand	95 percent	Near optimum

* Relative to ASTM Test D1557, latest edition.

Surface Drainage

Finished grades should be designed to prevent ponding of water and to direct surface water runoff away from foundations, edges of slabs and pavements, and toward suitable collection and discharge points. Slopes of at least 2 percent are recommended at buildings and foundation areas for the power block. Preferably, water discharged from roof downspouts and other storm drain systems should be collected in closed pipes that are routed to the storm drain system or other suitable discharge locations. Drainage facilities should be observed to verify that they are adequate and that no adjustments need to be made, especially during first two years following construction.

Drainage facilities should be periodically checked to verify that they are continuing to function properly. The drainage facilities will probably need to be periodically cleaned of silt and debris that may build up in the lines.

PRELIMINARY FOUNDATION RECOMMENDATIONS

The Project includes relatively heavy facilities in the power block areas, including the steam turbine/generator, cooling tower, and other equipment and tanks that will likely require mat foundation support possibly augmented by deep foundations where differential settlements are of concern. Lightly to moderately-loaded equipment and buildings likely can be supported on shallow foundations bearing on the native alluvial soil or compacted structural fill. Drilled pier foundations likely will be appropriate for the solar collector arrays, overhead piping, and on-site and off-site electrical infrastructure. The preliminary foundation design criteria will be updated when exploratory borings are advanced within the Project limits and along the alignment of the off-site linears during the design-level geotechnical investigation.

Shallow Foundations

Lightly to moderately-loaded buildings and equipment may be supported on continuous and isolated foundations bearing on undisturbed stiff or dense native soils or compacted structural fill. On a preliminary basis, footings should have a width of at least 24 inches and should extend at least 24 inches below exterior grade and at least 24-inches below the bottom of slab elevation, whichever is deeper. Footings with at least these minimum dimensions may be designed for an allowable bearing pressure of 2,500 pounds per square foot for combined dead plus live loads, with a one-third increase allowed when considering additional short-term wind or seismic loading. On-site drilling for the design level geotechnical investigation will provide blow count and other information that will likely raise this allowable bearing pressure. The weight of the footings may be neglected for design purposes.

Lateral loads will be resisted by friction between the bottom of the foundations and the supporting subgrade. A coefficient of friction of 0.3 may be assumed on a preliminary basis assuming granular soil or fill is present below the foundations. In addition, lateral resistance may be provided by passive soil pressure acting against the sides of foundations cast neat in the foundation excavations or backfilled with compacted structural fill. An equivalent fluid pressure of 250 pounds per cubic foot may be used for passive soil resistance, where appropriate. The upper foot of passive soil resistance should be neglected where soil adjacent to the footing will be landscaped or subject to softening from rainfall and/or surface water runoff.

Thirty year differential settlement due to static loads is not expected to exceed ¾-inch along and between shallow foundations designed in accordance with the criteria presented above. The amount of total and differential settlement should be evaluated once structural loads are available and site-specific subsurface conditions are confirmed.

Mat Foundations

The steam turbine/generator, cooling tower, and other structures where differential settlement is a concern may be supported on reinforced concrete mat foundations bearing on a properly prepared and compacted soil subgrade or on structural fill. An allowable bearing pressure of 3,000 to 4,000 pounds per square foot is expected to be appropriate for combined dead plus live loads with a one-third increase allowed when considering additional short-term wind or seismic loading.

Mat foundations should be reinforced to provide structural continuity and to permit spanning of local irregularities. The coefficient of friction and passive soil pressure recommended above for shallow foundations may also be used for mat foundations.

Total and differential settlement of mat foundations depends on the size and stiffness of the mat, the structural load it supports, and the modulus of the supporting subgrade materials. Individual estimates of total and differential settlement will be developed during or after the design-level geotechnical investigation when structural loads and geometry are available. If estimated settlements are not tolerable, the mats could be supported with reinforced concrete piers or piles to reduce differential settlement.

Drilled Pier Foundations

It is expected that the solar collector arrays will be supported on reinforced concrete pier foundations laid out in a grid pattern across the collection area. Some of the overhead piping, and on-site and off-site electrical infrastructure, will also be supported on pier foundations. Resistance to lateral loading is expected to be the controlling factor for design of piers supporting some of the structures. Drilled piers are expected to be a practical foundation to construct and use for support of the solar collectors and overhead infrastructure and may also be used where total and differential settlement of shallow foundations or mat foundations exceed allowable equipment or structural tolerances. Recommended allowable vertical and lateral pier capacity will be developed during the design-level geotechnical investigation.

SLABS ON GRADE

Concrete floor slabs, walkways, and exterior flatwork should be at least 4 inches thick and should be constructed on at least 6 inches of properly prepared and compacted select fill or granular native soil. The minimum required thickness of building and structure floor slabs will be determined by the structural engineer based on structure loading and use of the slab. Exterior slabs-on-grade may be constructed with a thickened edge to improve edge stiffness where desired. We expect that reinforced slabs will perform better than unreinforced slabs. Consideration should be given to using a control joint spacing on the order of 2 feet in each direction for each inch of slab thickness.

In general, loose surface soils and/or expansive surface soils should be removed to at least 24-inches below the bottom of the slab and replaced with non-expansive select fill.

In areas where dampness of concrete floor slabs would be undesirable, such as within the administrative building, warehouse building, control building, and other building interiors, concrete floor slabs should be underlain by at least 4 inches of clean, free-draining gravel, such as ½- to ¾-inch clean crushed rock with no more than 5 percent passing the ASTM No. 200 sieve. Pea gravel should not be used for this capillary break material. The crushed rock layer should be densified and leveled with vibratory equipment. To reduce vapor transmission up through concrete floor slabs, the crushed rock section should be covered with a high-quality, UV-resistant membrane vapor barrier meeting the minimum ASTM E1745, Class C requirements, or better, and preferably should be placed directly below the floor slab. All seams and penetrations of the vapor barrier should be sealed in accordance with manufacturer's recommendations. The crushed rock layer may be considered as the upper portion of the select fill recommended above.

PAVEMENTS

We understand the Project will include asphalt concrete pavements for the access road to the Project site from Interstate Highway 10 and for the main traffic drives around and between each of the power block areas. Unpaved roads will be used between the solar collectors and for maintenance and construction access. Some Portland Cement Concrete (PCC) pavements may used locally for specific facilities or applications. Extensive cutting and filling is not anticipated.

The surface soils at the site are typically expected to consist of silty and clayey sands with some local areas of moderately to highly plastic clay. For this preliminary study, we selected an R-value of 40 for use in pavement thickness design where the pavements will be supported on competent granular native soil and a design R-value of 8 for areas underlain by soils with significant silt or clay content. Where expansive soils are exposed at pavement subgrade elevation, they should be excavated and removed to a depth of 2 feet or as directed by our representative in the field at the time of construction. The preliminary pavement thickness guidelines presented in this report will be updated after subsurface exploration and laboratory testing is performed for the design-level geotechnical investigation. After rough grading to pavement subgrade elevation is completed, the R-value of the subgrade should be further evaluated and the final pavement section thicknesses confirmed.

Asphalt Concrete Pavement

Using a range of Traffic Indices selected to simulate the currently anticipated traffic loading, we developed the minimum pavement section thicknesses presented in Tables 4 and 5 on the following page based on Procedure 608 of the Caltrans Highway Design Manual. The minimum pavement section thicknesses shown on Table 4 assume the pavement subgrade will be composed of competent sandy native or imported soil with an R-value of at least 40. The pavement section thicknesses shown on Table 5 assume the pavement subgrade will be composed of native clay or imported soil with an R-value of at least 8.

The Traffic Indices used in our pavement thickness calculations are based on engineering judgment rather than on a detailed analysis of future pavement loading conditions.

Asphalt concrete and aggregate base should conform to and be placed in accordance with the requirements of the Caltrans Standard Specifications, latest edition, except that compaction should be based on ASTM Test D1557.

Table 4. Minimum Asphalt Concrete Pavement Thicknesses

Design R-value = 40 (Competent Granular Soil Subgrade)

General Traffic Condition	Traffic Index (inches)	AC Thickness (inches)	Aggregate Base* (inches)	Total Section (inches)
Automobile Traffic Lanes	4.5	3.0	4.0	7.0
Truck Traffic	5.5	3.0	6.0	9.0
	6.0	3.0	7.0	10.0
	7.0	4.0	7.0	11.0
	8.0	4.0	9.0	13.0
	9.0	5.0	10.0	15.0

* Caltrans Class 2 Aggregate Base (minimum R-value = 78).

Table 5. Minimum Asphalt Concrete Pavement Thicknesses

Design R-value = 8 (Subgrade Soils With Significant Silt Or Clay Content)

General Traffic Condition	Traffic Index (inches)	AC Thickness (inches)	Aggregate Base* (inches)	Total (inches)
Automobile Traffic Lanes	4.5	3.0	8.0	11.0
Truck Traffic	5.5	3.0	11.0	14.0
	6.0	3.0	13.0	16.0
	7.0	4.0	15.0	19.0
	8.0	4.0	18.0	22.0
	9.0	5.0	20.0	25.0

***** Caltrans Class 2 Aggregate Base (minimum R-value = 78).

We recommend that measures be taken to limit the amount of surface water that seeps into the aggregate base and subgrade below vehicle pavements, particularly where the pavements are adjacent to landscaping. Seepage of water into the pavement base can soften the subgrade, thereby increasing the amount of pavement maintenance that is required and shortening the pavement service life. Deepened curbs extending at least 4-inches into the subgrade below the aggregate base and subbase layers are generally effective in limiting excessive water seepage. Other types of water cutoff devices or edge drains may also be considered to maintain pavement service life.

Portland Cement Concrete Pavement

If Portland Cement Concrete (PCC) pavement will be used in areas to be occasionally driven on by fire trucks or other heavy vehicles, we recommend the PCC pavement section be constructed at least 7 inches thick on 6 inches of compacted Class 2 aggregate base on a properly prepared and compacted subgrade. This pavement section thickness is based on guidelines published by the Portland Cement Association and assumes that concrete for the pavement will have a modulus of rupture of 550 psi, which roughly corresponds to a concrete compressive strength of 3,700 psi. Concrete pavement should have adequate construction joints and crack control joints. If concrete pavements will be used for this project, the geotechnical engineer should work with the design engineer during the design-level geotechnical investigation to provide additional information and alternatives for concrete pavements based on the expected traffic loading conditions for the Project site.

CORROSION POTENTIAL TESTING

Corrosion potential tests were performed on three samples of surface soil obtained across the Project site. All of the samples were tested for resistivity, pH, chloride content, sulfate content, and oxidation-reduction potential (ORP).

Resistivity of the lab-saturated soil samples measured in accordance with ASTM Test G57 ranged from 11,540 to 16,450 ohm-cm. ASTM STP 1013 titled “Effects of Soil Characteristics on Corrosion” indicates soil resistivity of 10,000 to >100,000 ohm-cm would classify soil as very mildly corrosive.

The pH of the soil samples ranged from 7.9 to 8.1. A pH between 5 and 8.5 is generally considered relatively passive from a corrosion standpoint. As pH increases, the soil is considered more alkaline and less corrosive. Chloride content was less than 2 mg/kg (ppm). The oxidation-reduction potential (Redox) ranged from 128 to 188 mv.

The water-soluble sulfate content of the samples that were tested in accordance with California Test Method 417-modified were measured to be <5 parts per million (<0.0005% by dry weight). Table 19A-A-4 of the California Building Code classifies a water-soluble sulfate content of 0.0 to 0.10% by dry weight as producing negligible sulfate exposure.

The results of the corrosion potential tests should be considered preliminary and additional testing should be performed during the design-level geotechnical investigation to further investigate the corrosion potential of the on-site soils. After supplemental corrosion potential testing, a corrosion specialist could be consulted for a more complete analysis and additional design recommendations.

FUTURE SERVICES

At this time, permission has not yet been granted by the Bureau of Land Management to conduct geotechnical borings and surface and subsurface exploration at the site for the proposed improvements. Once the site is accessible, and once the need for further data to support detailed design is required, this preliminary study should be updated with on-site exploration, laboratory testing, and analysis of specific structure foundations in order to complete a design-level geologic and geotechnical investigation for the project to support the detailed design of the project.

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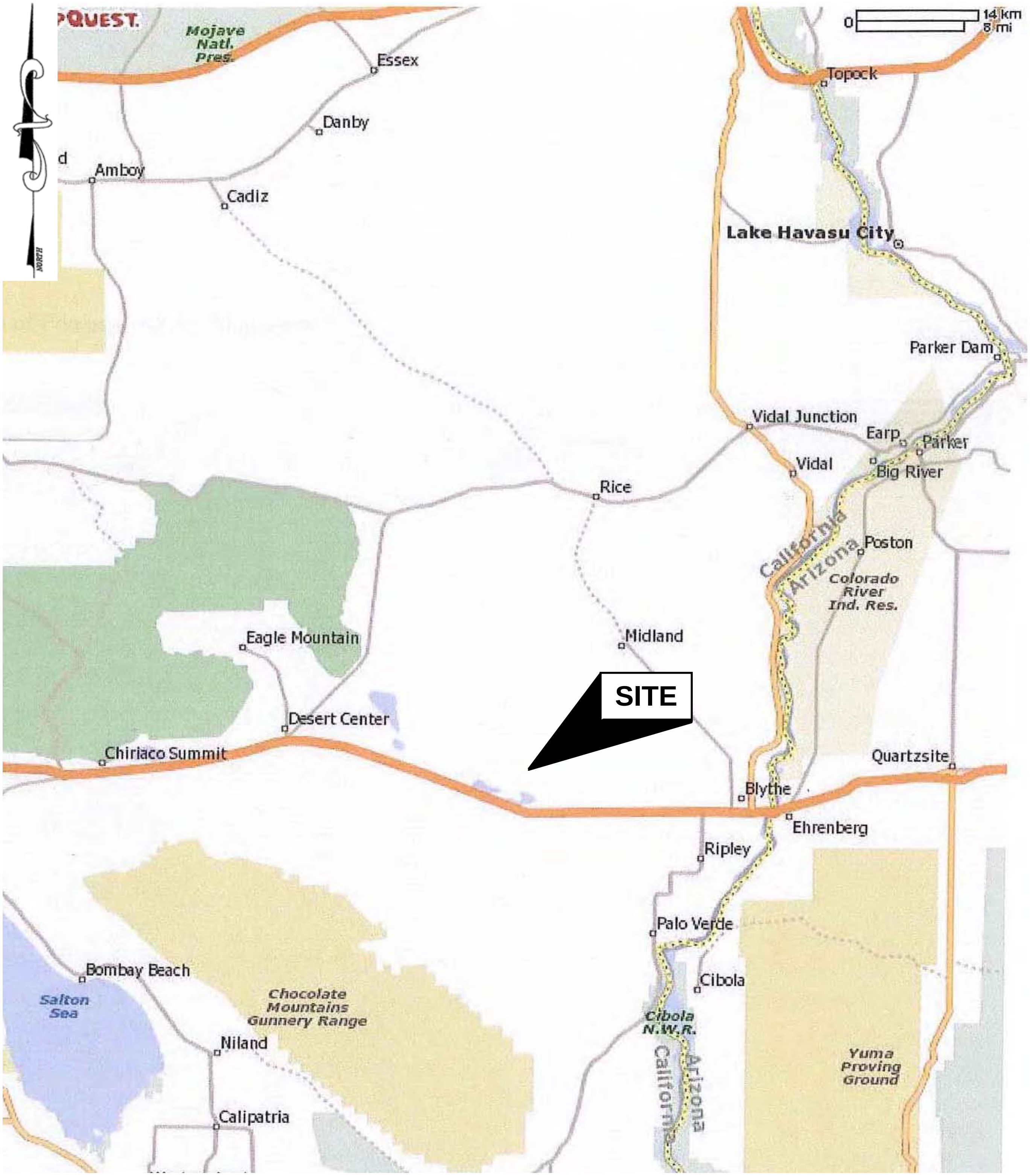
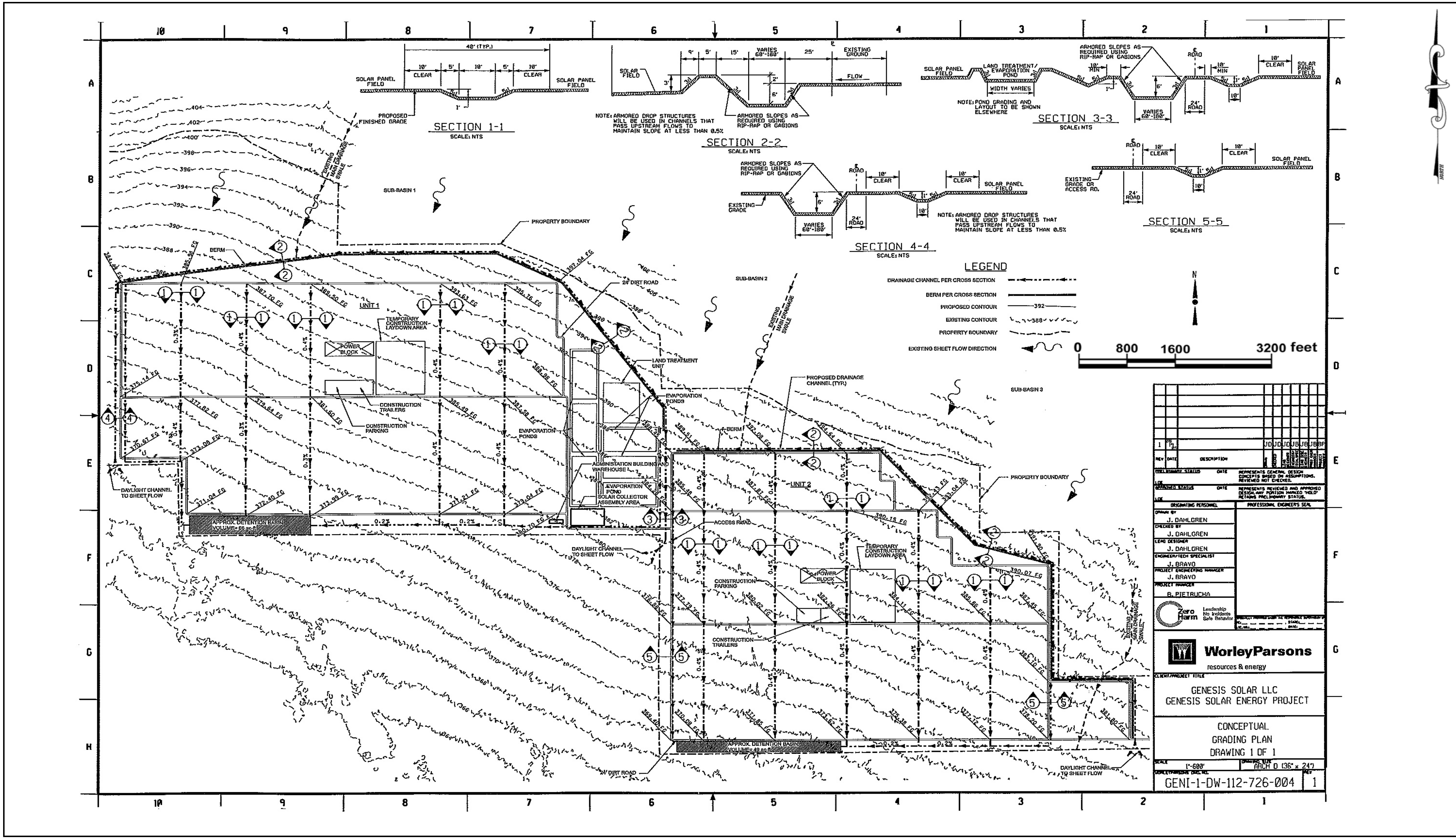


Base from MapQuest Inc, 2009.

VICINITY MAP **FIGURE 1**

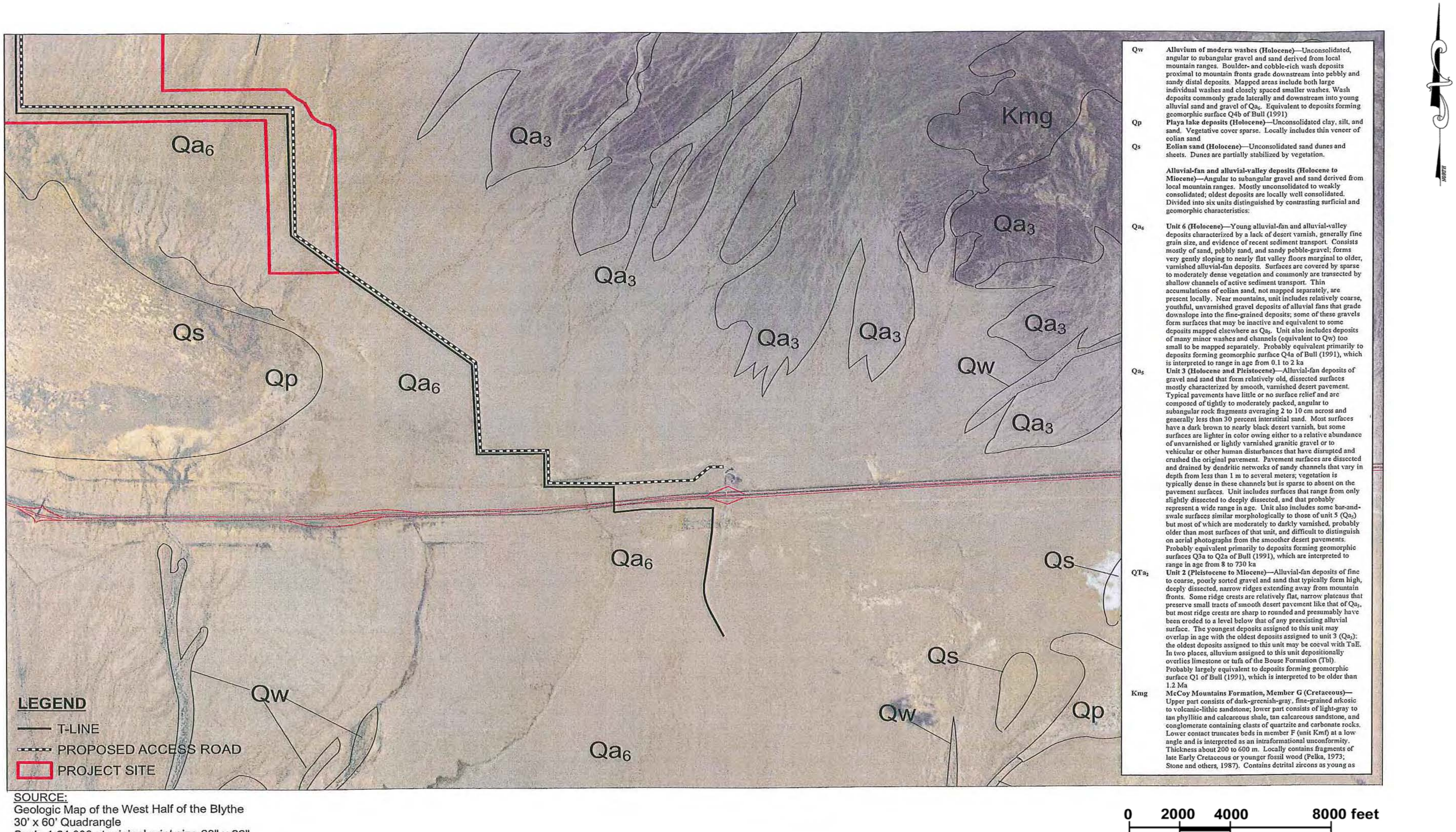
GENESIS SOLAR ENERGY PROJECT OCTOBER 2009
CHUCKWALLA VALLEY PROJECT NO. 2341-1
RIVERSIDE COUNTY, CALIFORNIA



SITE PLAN
GENESIS SOLAR ENERGY PROJECT
CHUCKWALLA VALLEY
RIVERSIDE COUNTY, CALIFORNIA

FIGURE 2
OCTOBER 2009
PROJECT NO. 2341-1

ROMIG ENGINEERS, INC.



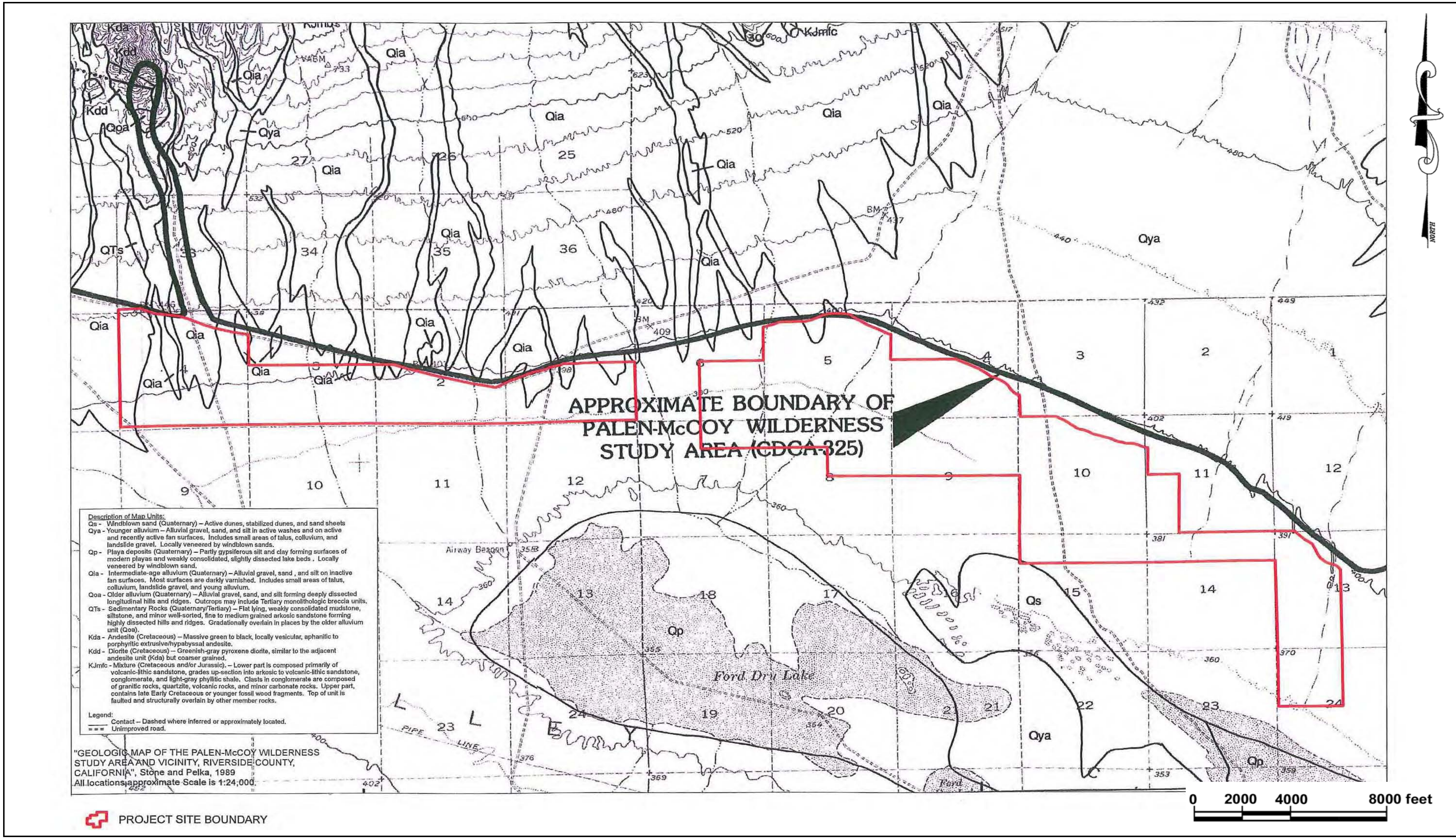
SOURCE:
Geologic Map of the West Half of the Blythe 30' x 60' Quadrangle
Scale 1:24,000 at original print size 28" x 22"
All locations approximate

AREA GEOLOGIC MAP FOR OFF-SITE LINEARS **FIGURE 3**

GENESIS SOLAR ENERGY PROJECT
CHUCKWALLA VALLEY
RIVERSIDE COUNTY, CALIFORNIA

OCTOBER 2009
PROJECT NO. 2341-1

ROMIG ENGINEERS, INC.

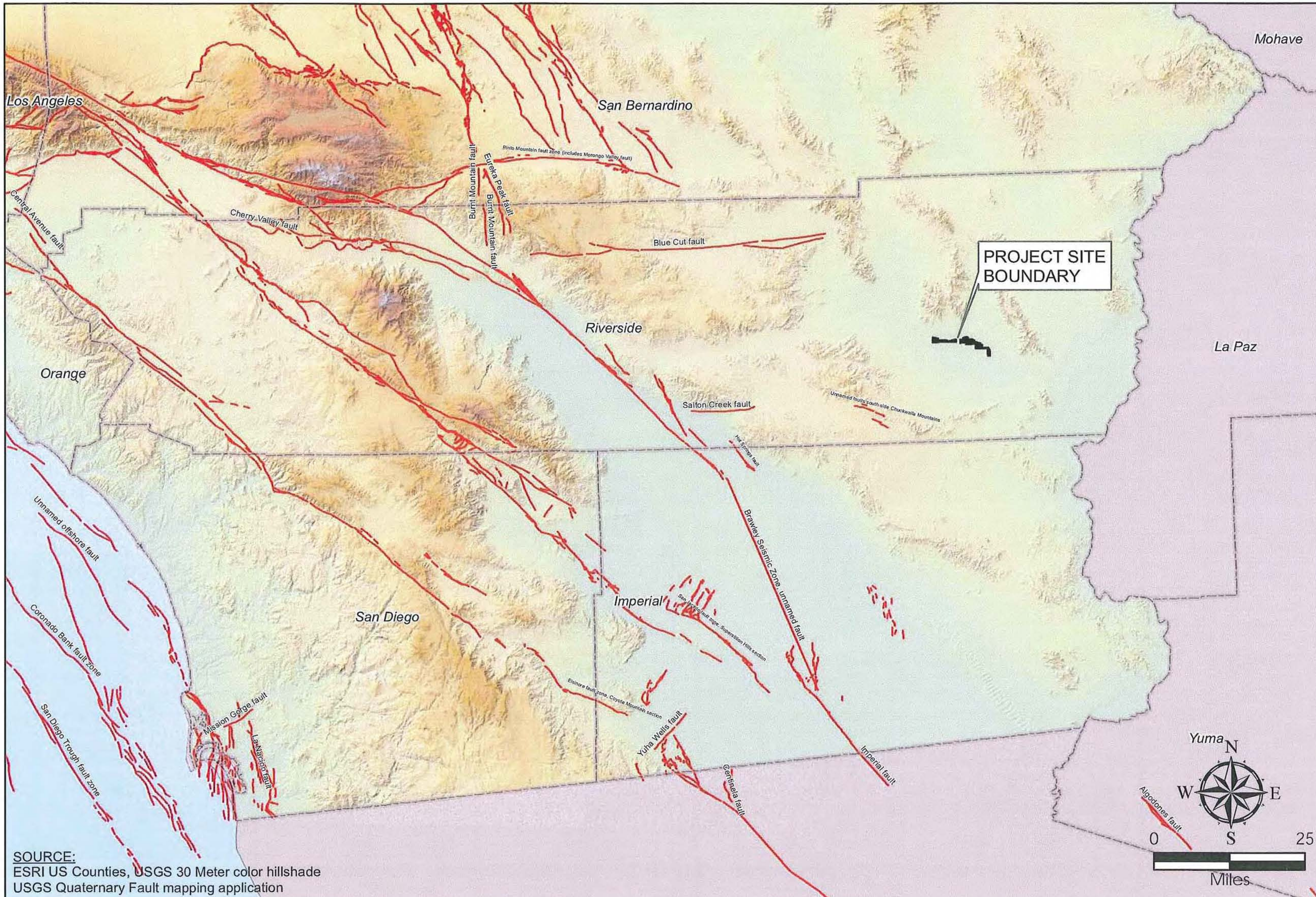


AREA GEOLOGIC MAP **FIGURE 4**

GENESIS SOLAR ENERGY PROJECT
CHUCKWALLA VALLEY
RIVERSIDE COUNTY, CALIFORNIA

OCTOBER 2009
PROJECT NO. 2341-1

ROMIG ENGINEERS, INC.

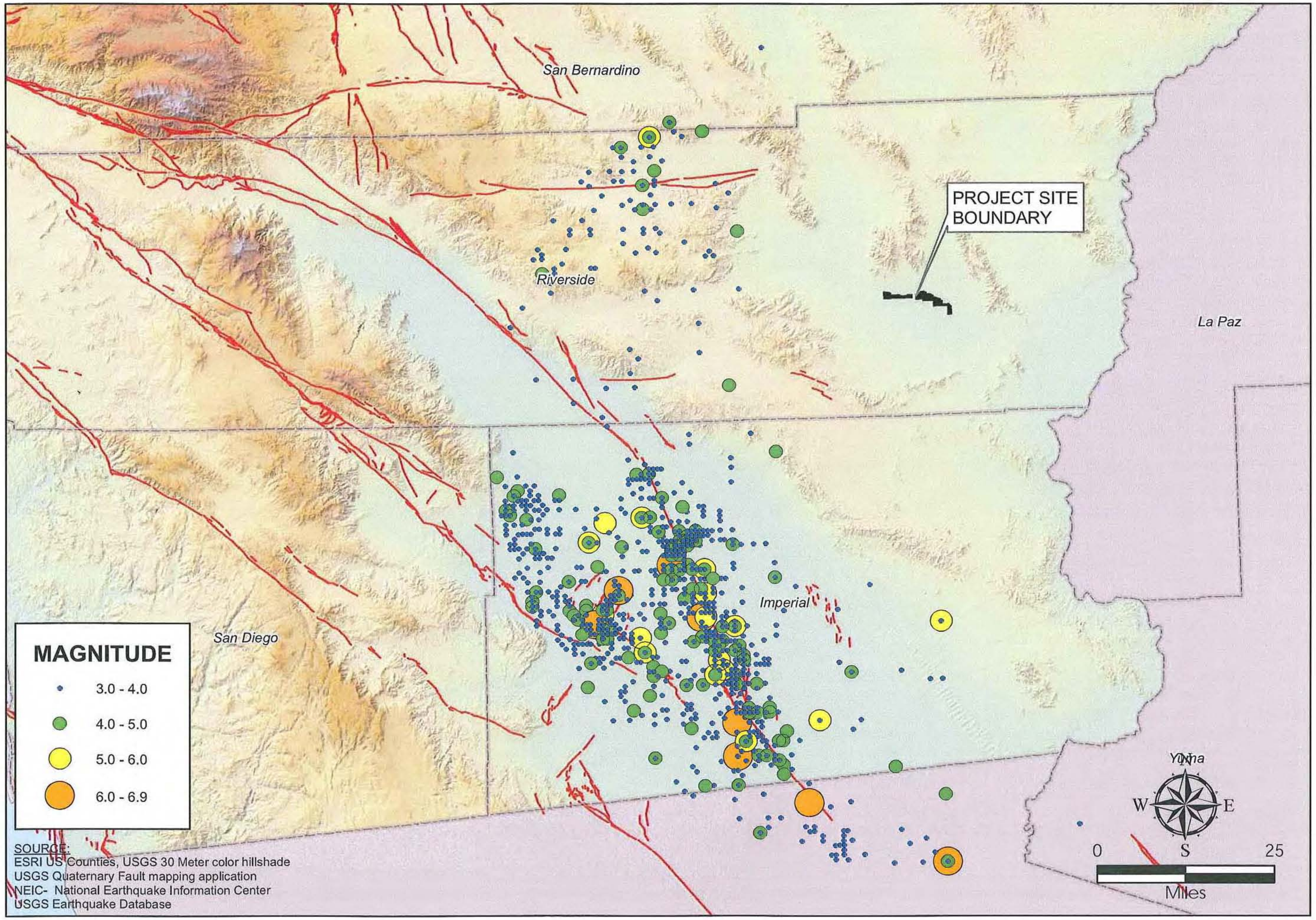


REGIONAL FAULT LOCATION MAP

GENESIS SOLAR ENERGY PROJECT
CHUCKWALLA VALLEY
RIVERSIDE COUNTY, CALIFORNIA

FIGURE 5

OCTOBER 2009
PROJECT NO. 2341-1

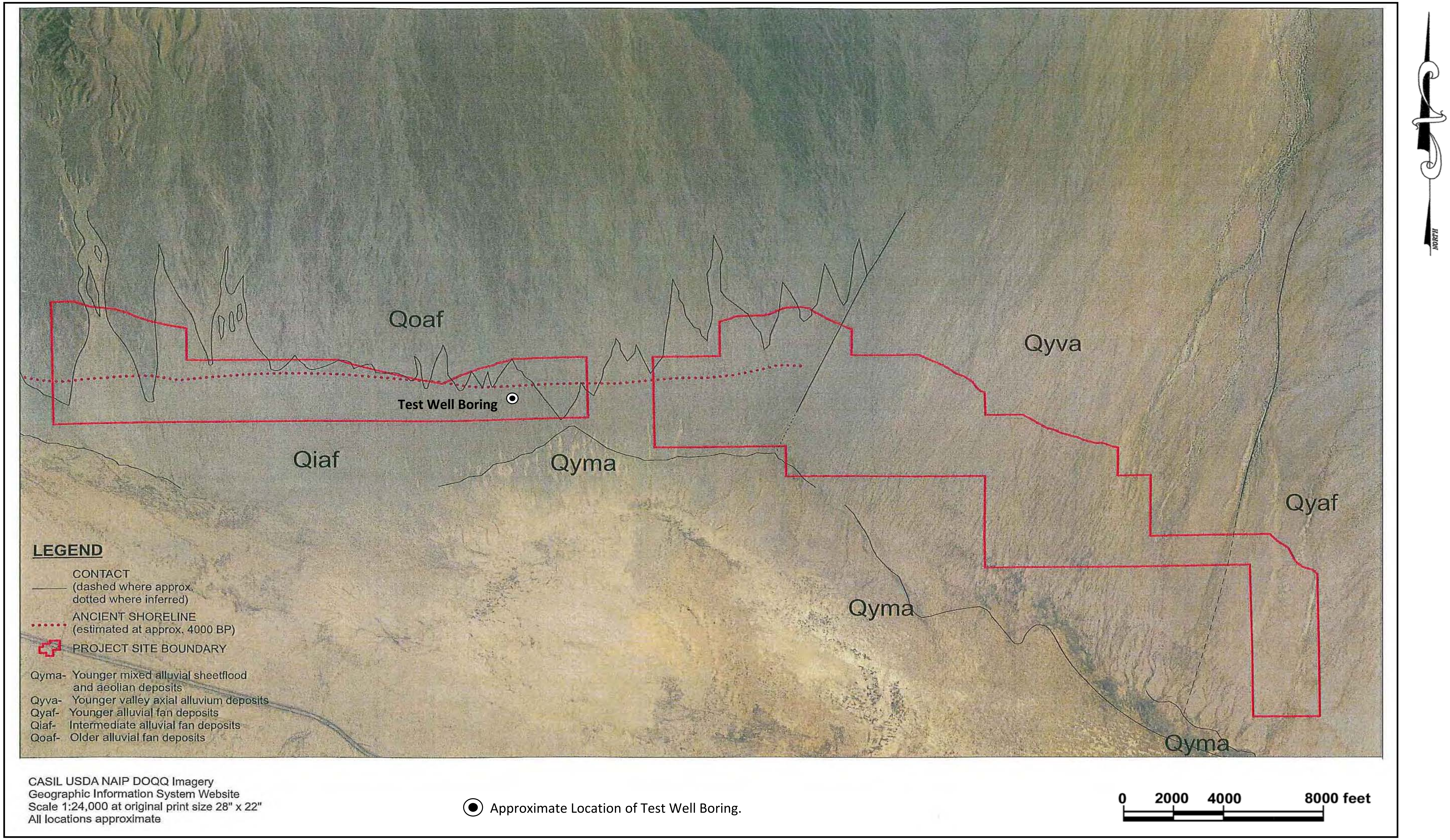


HISTORICAL EARTHQUAKES WITHIN 100 KM FROM THE SITE

FIGURE 6

GENESIS SOLAR ENERGY PROJECT
CHUCKWALLA VALLEY
RIVERSIDE COUNTY, CALIFORNIA

OCTOBER 2009
PROJECT NO. 2341-1



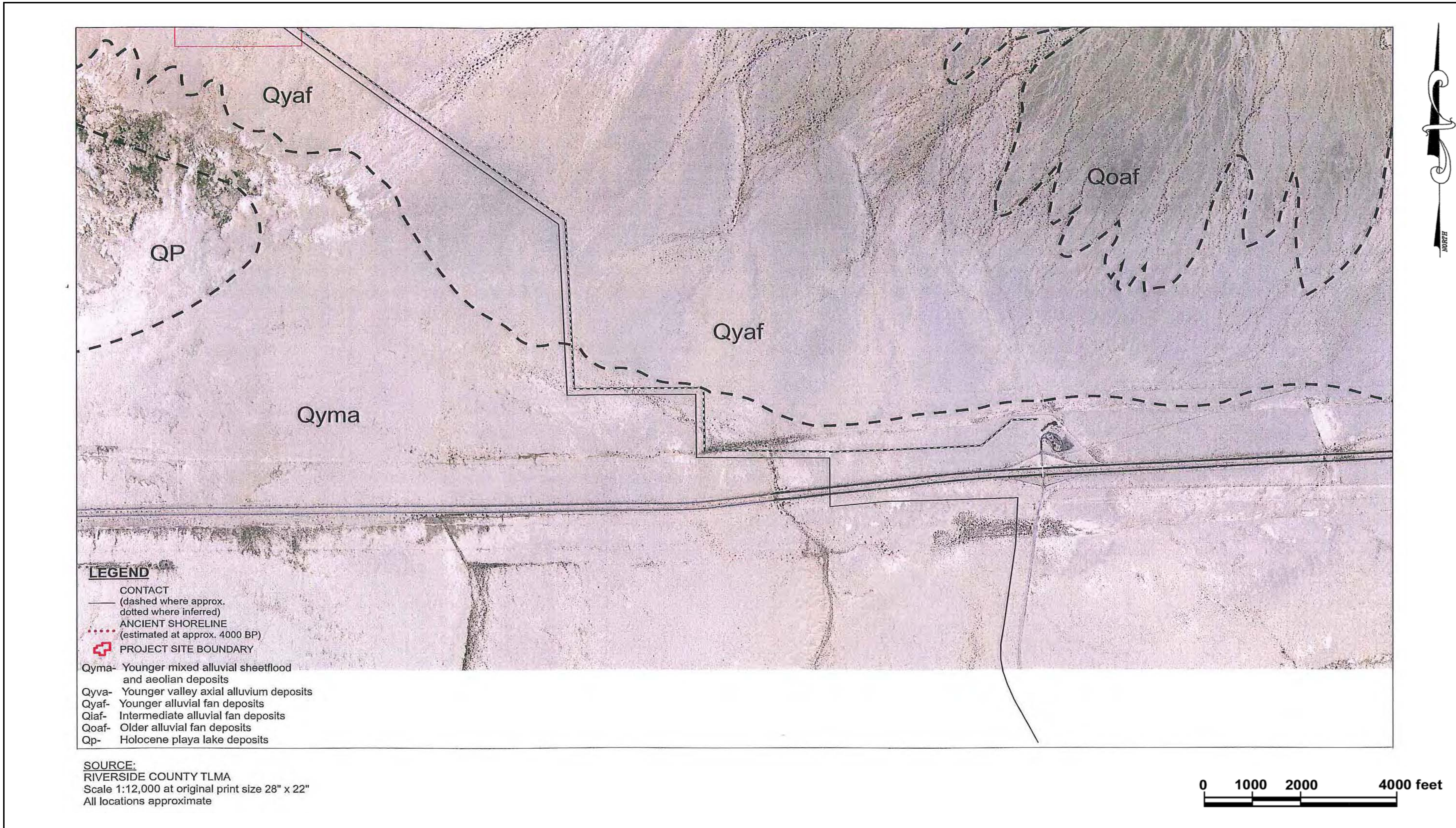
SITE GEOLOGIC MAP | **FIGURE 7**

GENESIS SOLAR ENERGY PROJECT | OCTOBER 2009

CHUCKWALLA VALLEY | PROJECT NO. 2341-1

RIVERSIDE COUNTY, CALIFORNIA

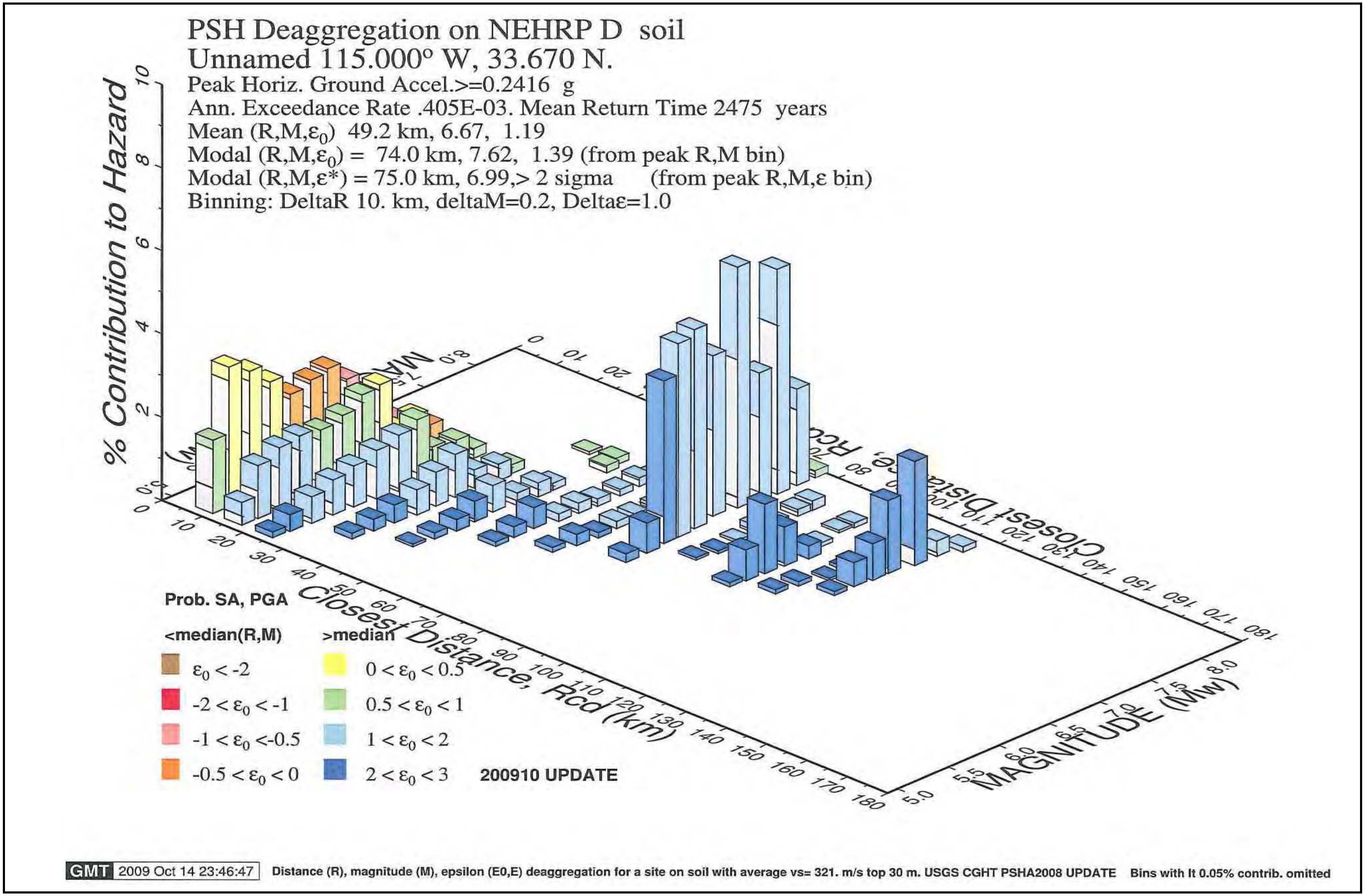
ROMIG ENGINEERS, INC.



SITE GEOLOGIC MAP FOR OFF-SITE LINEARS | **FIGURE 8**

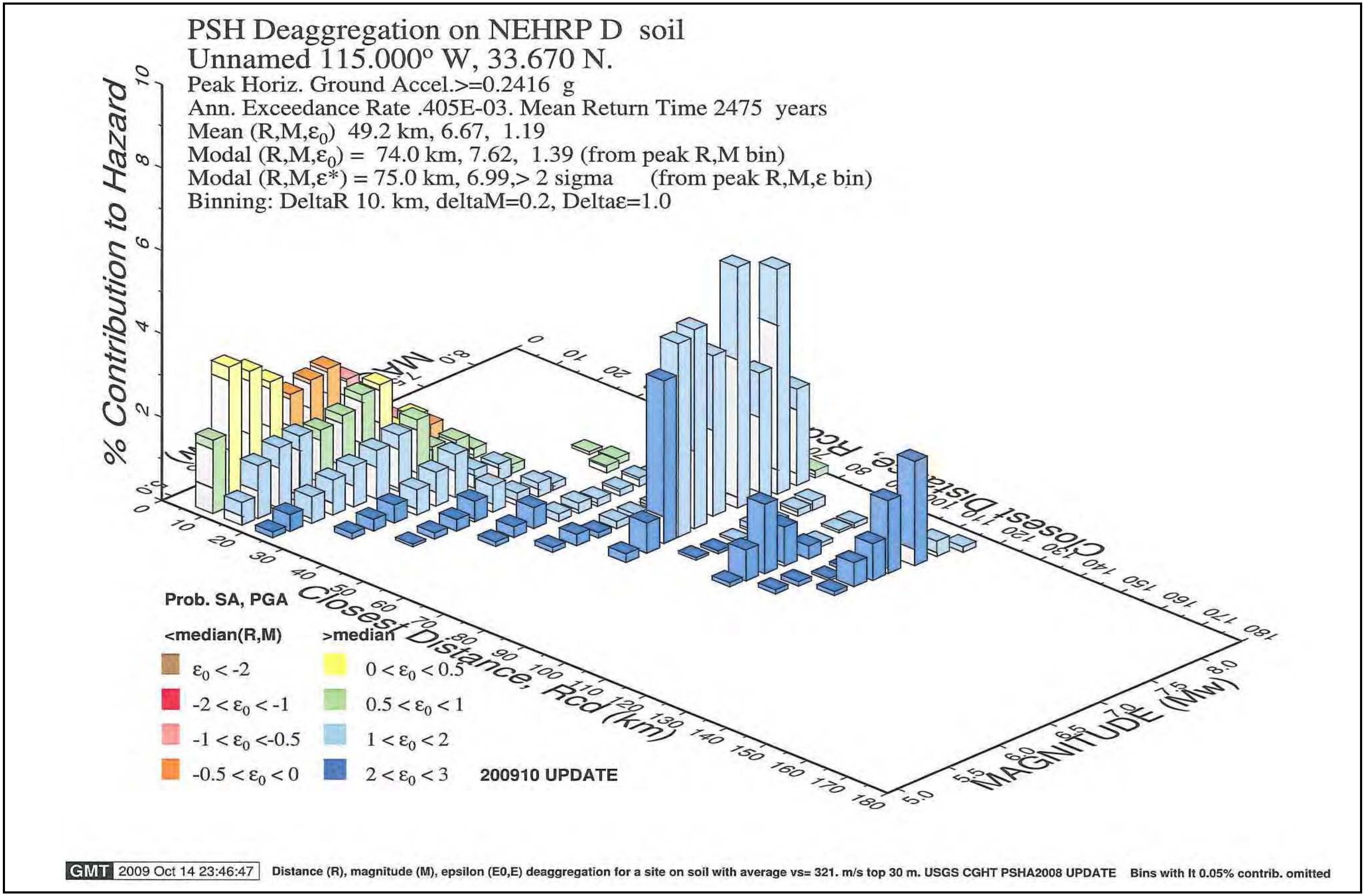
GENESIS SOLAR ENERGY PROJECT
CHUCKWALLA VALLEY
RIVERSIDE COUNTY, CALIFORNIA

OCTOBER 2009
PROJECT NO. 2341-1



DEAGGREGATION DATA/475-YEAR RETURN PERIOD
GENESIS SOLAR ENERGY PROJECT
CHUCKWALLA VALLEY
RIVERSIDE COUNTY, CALIFORNIA

FIGURE 9
OCTOBER 2009
PROJECT NO. 2341-1



DEAGGREGATION DATA/2475-YEAR RETURN PERIOD **FIGURE 10**

GENESIS SOLAR ENERGY PROJECT OCTOBER 2009

CHUCKWALLA VALLEY PROJECT NO. 2341-1

RIVERSIDE COUNTY, CALIFORNIA

APPENDIX A

FIELD AND LABORATORY DATA

Lithologic Log of OBS-2 - 0 fbgs to 75 fbgs

Genesis Solar, LLC

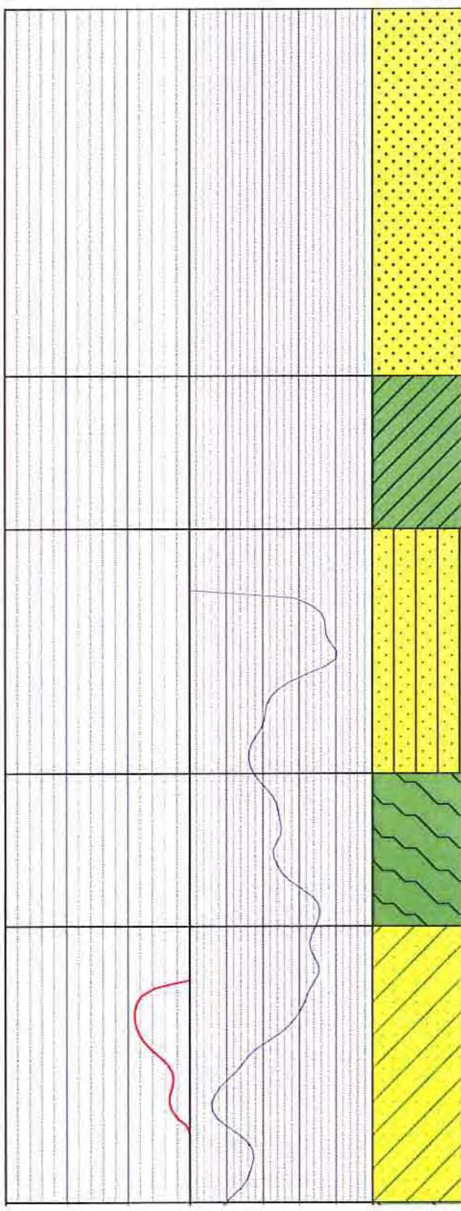
Project Number: 52004617

Date Drilled: 05/28/2009 to 07/02/2009		Borehole Location: N33°40.419' W115°03.268	
Drilling Method: Dual Tube Reverse Circulation		Ground Surface Elevation: 383 feet amsl	
Drilling Contractor: WDC Exploration		Static Water Level: N/A	
Geologist: Andie Gehlhausen	Reviewer: Nat Beal	Total Depth: 900 ft	Logged Depth: 80 ft

Notes:
1) Lithologic log was adjusted based on the cuttings log from OBS-1 and the geophysical logs
2) RSN and RLS have been corrected to 77 degrees F
3) Soil samples were collected using a Modified California Split Spoon Sampler and a standard 140-pound drive hammer

GEOPHYSICAL LOGS: RLN (OHM-M) 0–15; RSN (OHM-M) 0–15; Gamma (GAPI) 40–140

Depth - Feet; Blows (% Recovey); Sample Interval



Depth - Feet	Blows (% Recovey)	USCS Soil Type	Geologic Description	Remarks
0		SW	GRAVELLY SAND WITH SILT: Yellowish brown (10YR 5/4), very dense, well graded fine to coarse grained sand, subangular to surounded fine to medium grained gravel, dry to slightly moist.	
	100 for 6" (33 %)			16 % passing No. 200
10	100 for 6"			17 % passing No. 200
	100 for 18"	CL	SANDY LEAN CLAY: Yellowish brown (10YR 5/4), hard, low to moderate plasiticy clay, fine to coarse grained sand, trace fine subangular gravel, trace carbonate cementation, moist.	
20	100 for ? (22 %)	SM	SILTY SAND: Yellowish brown (10YR 5/4), very dense, well graded fine to coarse grained sand, trace fine subangular to subrounded gravel, moist.	
	100 for ? (28 %)	CH	FAT CLAY: Yellowish brown (10YR 5/4) with black mottling, hard, moderate to high plasiticy clay, moist.	Torvane Shear Strength: 2.5 ksf. LL = 58, PL = 37, Free Swell = 130 %
30	75 for 6" and 100 for 0" (61 %)	CL/SC	Interbedded SANDY LEAN CLAY and CLAYEY SAND: Yellowish brown (10YR 5/4), hard, low to moderate plasiticy clay with fine sand, yellowish brown (10YR 5/4), very dense, fine grained clayey sand, moist.	Unconfined Compressive Strength: 3.75 ksf , Torvane Shear Strength: 2.5 ksf
	68 for 6" and 100 for 0" (33 %)			Unconfined Compressive Strength: 3.25 ksf, Torvane Shear Strength: 3.75 ksf

Lithologic Log of OBS-2 - 0 fbgs to 75 fbgs

WorleyParsons
resources & energy

Genesis Solar, LLC
Project Number: 52004617

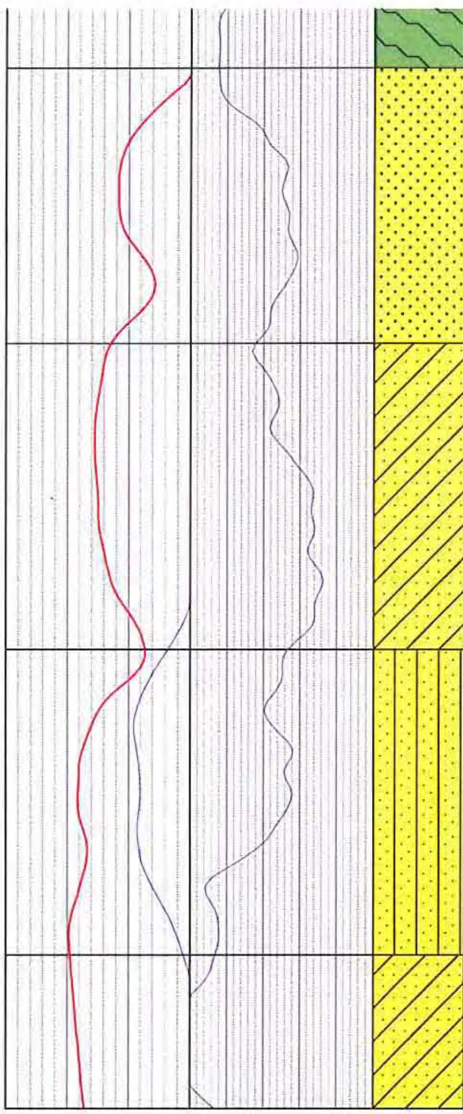
Date Drilled: 05/28/2009 to 07/02/2009		Borehole Location: N33°40.419' W115°03.268	
Drilling Method: Dual Tube Reverse Circulation		Ground Surface Elevation: 383 feet amsl	
Drilling Contractor: WDC Exploration		Static Water Level: N/A	
Geologist: Andie Gehlhausen	Reviewer: Nat Beal	Total Depth: 900 ft	Logged Depth: 80 ft

Notes:
1) Lithologic log was adjusted based on the cuttings log from OBS-1 and the geophysical logs
2) RSN and RLS have been corrected to 77 degrees F
3) Soil samples were collected using a Modified California Split Spoon Sampler and a standard 140-pound drive hammer

GEOPHYSICAL LOGS

Depth - Feet | Blows (% Recovey) | RLN (OHM-M) 0 – 15 / RSN (OHM-M) 0 – 15 | Gamma (GAPI) 40 – 140 | Graphic Log | USCS Soil Type | Geologic Description | Remarks

Sample Interval



Depth - Feet	Blows (% Recovey)	USCS Soil Type	Geologic Description	Remarks
40	68 for 6" and 100 for 0" (33 %)	CH	FAT CLAY: Yellowish brown (10YR 5/4), hard, moderate to high plasticity clay, moist.	Free Swell = 80 %
	41 for 6" and 100 for 11"	SW	SAND WITH SILT: Yellowish brown (10YR 5/4), very dense, fine sand with approximately 20 % silt, moist	
50	62 for 6", 87 for 12" & 100 for 0" (25 %); 59 for 6" and 100 for 10"	SC	CLAYEY SAND Yellowish brown (10YR 5/4), very dense, fine sand with laminated, moderate plasticity fines, moist	
60	61 for 6" and 100 for 7" (28 %); 65 for 6" and 100 for 7" (22 %)	SM	SILTY SAND: Yellowish brown (10YR 5/4), very dense, fine grained sand, with lenses of SANDY LEAN CLAY: yellowish brown (10YR 5/4), hard, moderate plasticity clay, moist	48 % passing No. 200; LL = 40, PL = 23, Free Swell = 30 %
70	22 for 6", 34 for 12" & 100 for 14" (17 %)	SC	CLAYEY SAND Yellowish brown (10YR 5/4), very dense, fine sand, laminated, with moderate plasticity fines, moist	Unconfined Compressive Strength: 1.5 ksf

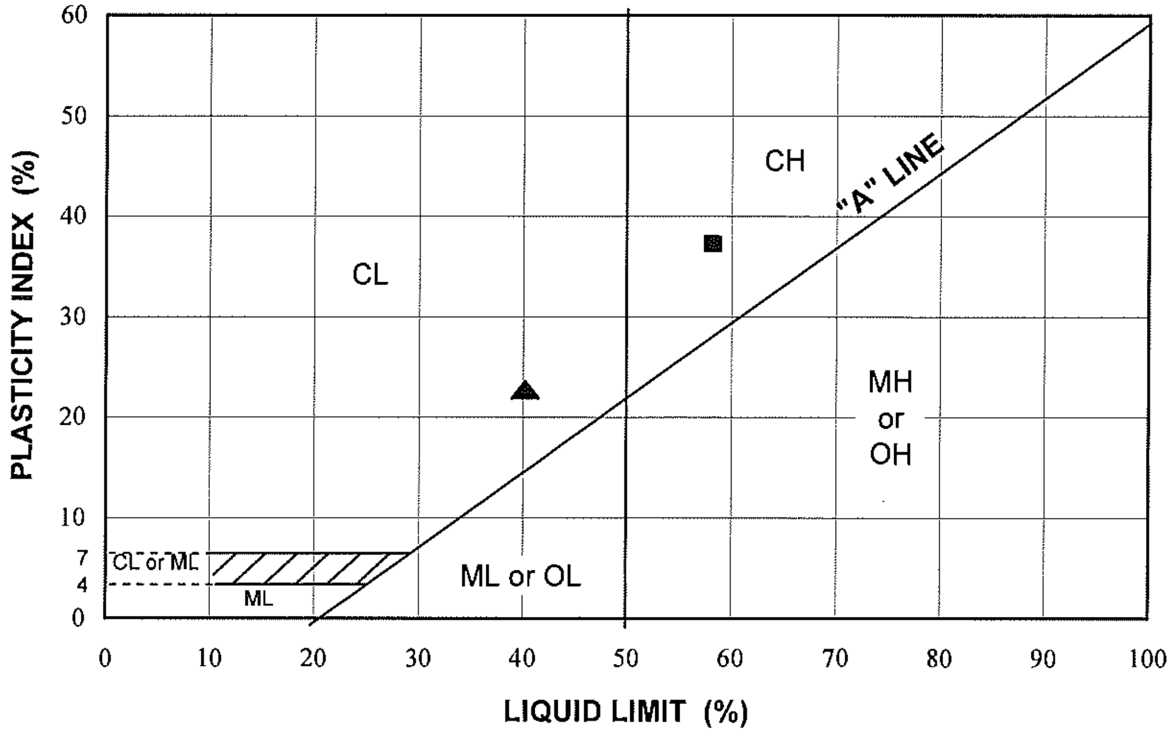


Chart Symbol	Boring Number	Sample Depth (feet)	Water Content (percent)	Liquid Limit (percent)	Plasticity Index (percent)	Liquidity Index (percent)	Passing No. 200 Sieve (percent)	USCS Soil Classification
■	EB-1	25	21	58	37			CH
▲	EB-1	65	16	40	23			CL

PLASTICITY CHART **FIGURE A-1**
GENESIS SOLAR ENERGY PROJECT OCTOBER 2009
CHUCKWALLA VALLEY PROJECT NO. 2341-1
RIVERSIDE COUNTY, CALIFORNIA

COOPER TESTING LABORATORY

Corrosivity Test Summary

CTL #: 192-128   Date: 8/20/2009   Tested By: PJ   Checked: PJ

Client: Romig Engineers   Project: Worley Parsons   Proj. No: 2341-1

Remarks:

Sample Location or ID			Resistivity @ 15.5 °C (Ohm-cm)			Chloride	Sulfate-(water soluble)		pH	ORP	Sulfide	Moisture	Soil Visual Description
Boring	Sample, No.	Depth, in.	As Rec.	Minimum	Saturated	mg/kg	mg/kg	%		(Redox)	Qualitative	%	
						Dry Wt.	Dry Wt.	Dry Wt.		mv	by Lead	At Test	
			ASTM G57	Cal 643	ASTM G57	Cal 422-mod.	Cal 417-mod.	Cal 417-mod.	ASTM G51	SM 2580B	Acetate Paper	ASTM D2216	
-	2	0-8"	-	-	11,540	<2	<5	<0.0005	7.9	128	-	0.6	Dark Yellowish Brown Silty SAND
-	3	0-8"	-	-	16,450	<2	<5	<0.0005	8.1	188	-	0.3	Dark Yellowish Brown Silty SAND
-	4	0-8"	-	-	12,953	<2	<5	<0.0005	8.1	164	-	0.3	Dark Yellowish Brown Silty SAND

APPENDIX B

GEOPHYSICAL SURVEY

J R ASSOCIATES

Engineering Geophysics
1886 Emory Street
San Jose, CA 95126
(408) 293-7390

GEOPHYSICAL SHEAR WAVE INVESTIGATION AT FORD DRY LAKE NEAR BLYTHE IN RIVERSIDE COUNTY, CALIFORNIA

September 21, 2009

for

WorleyParsons Group, Incorporated
2330 E Bidwell Street, Suite 150
Folsom, CA 95630

by

James Rezowalli
California Registered Geophysicist, GP-921

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I INTRODUCTION

This report presents the results of a geophysical shear (S-) wave investigation performed north of Ford Dry Lake near Blythe in Riverside County, California. The investigation was performed for WorleyParsons Group, Incorporated, by J R Associates. The objectives of the investigation were:

> Conduct a downhole shear wave test at the shallow observation well installed at the test well cluster to look for low shear wave velocities that are an indication of weak soil zones.
>
> Collect shear wave velocity profiles at three locations using the Multichannel Analysis of Surface Waves (MASW) method. Compare MASW results to downhole shear wave data. Look for low velocity shear wave zones indicative of weak soils under the three MASW traverses.

James Rezowalli, Principal Geophysicist, Garret Rhett, Technician, and Jeff Spackman, Technician, of J R Associates performed the field work in September of 2009.

A. Site Conditions

The area of interest is just north of Ford Dry Lake approximately 20 miles west of Blythe, California (Drawing 1). The site consists of dry flat desert and dry lake bed. Lithologic logs from test wells at the site indicate the upper 75 feet of soil is a younger alluvium containing a mixture of sands, silts, and clays. The water table at the site is approximately 75 feet below grade.

Genesis Solar LLP proposes to develop a power plant at the site. Information on compressible or liquefiable soils was needed for the project. Studies have shown a relationship between shear wave velocities and liquefaction resistance of soils[1]. In general soils with low shear wave velocities are more prone to liquefaction than soils with higher shear wave velocities. Because most of the site is only accessible by foot and motor vehicles are prohibited, conventional methods for determining soil strength, such as a cone penetrometer or a standard penetration test, were not allowed. The MASW method of collecting shear wave velocity profiles was chosen because it could be performed on foot in areas presently inaccessible to drill rigs. Shear wave data were also collected in an existing observation well.

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[1]Andrus, R.D. and Stokoe, K.H. (2000), "Liquefaction Resistance of Soils From Shear Wave Velocity." *Journal of Geotechnical and Geoenvironmental Engineering*, Vol 126, No 11, November 2000, 1015-1025.

II METHODOLOGY

We used two geophysical methods in our investigation, downhole compressional (P-) and shear (S-) wave measurements and the multichannel analysis of surface wave method (MASW). Drawing 2 illustrates the two methods. The downhole method involves creating P- and S-waves on the surface and measuring their travel times to a receiver in a borehole. From a graph of travel times versus depth, P- and S- wave velocities for the soil adjacent to the borehole are calculated creating a one-dimensional velocity profile. The MASW method involves measuring the dispersion of a surface wave created at one end of a string of receivers. From the dispersion data a one dimensional S-wave velocity profile is calculated. By collecting several profiles along a traverse, a two-dimensional shear wave profile can be created.

A. Downhole Field Procedures and Instrumentation

Two downhole P- and S-wave profiles were collected in the shallow observation well at the test well cluster (Drawing 3). We began data collection by installing a P-wave and an S-wave source on the ground near the borehole. The P-wave source consisted of a 12-pound sledge hammer striking an aluminum plate. The S-wave source consisted of a 4x4 wooden beam laying on its side on the ground. We drove a truck onto the beam to hold it in place. One end of the beam was struck with the sledge hammer to create an S-wave. We could change the polarization of the S-wave by striking the other end of the beam. S-waves are picked from a seismograph recording by looking for a standout in amplitude and the polarity reversal in the recorded wave forms.

At the start of a test a triaxial geophone was lowered to the bottom of a borehole and locked to a borehole wall. We then generated a P-wave and a pair of S-waves on the ground surface and recorded their arrivals at the geophone. The S-wave pair consisted of a forward polarized wave and a reversed polarized wave. We then raised the geophone 5 feet and collected another set of waves. This process was repeated until the geophone was 5 feet from the ground surface. We collected two sets of data, one with the sources ten feet from the borehole and the other with the sources fifteen feet from the borehole.

A Litton LRS-1023 triaxial geophone was lowered into the borehole to detect the seismic signals. A cable connected the geophones to a Geometrics Geode seismograph which in turn was connected to a personal computer. The computer filtered, stacked, and recorded the signals. Stacking (adding) signals from multiple hammer blows at the same source point improves the signal to noise ratios of the recordings. Typically four recordings at each geophone depth and source were stacked.

Data reduction began by picking the arrival times from the seismograph recordings. An arrival time is the time a wave spent traveling from a source point to the geophone. The waves were assumed to travel in a straight line from the source to the triaxial geophone. The arrival times versus depths were plotted and the P- and S-wave velocities were calculated from the plot. We calculated small strain values of Poisson's ratio and shear modulus from the averaged P- and S-wave velocities. A unit weight of 110 pounds per cubic foot was assumed for the shear modulus calculation.

B. <u>MASW Field Procedures and Instrumentation</u>

MASW data were collected along a test line adjacent to the well cluster and along three 294-foot profile lines on the eastern side of the site (Drawing 3). Data were collect along the test line to establish the optimum shot point offset and to compare the MASW and downhole results.

MASW data collection began by placing the plate 30 feet from the end of a string of 24 geophones. The geophones were spaced three feet apart. Surface waves were created by striking an aluminum plate and the waves were recorded. Once a multichannel record was collected, the plate and geophone array were advanced 15 feet along the line and the process was repeated. A total of fourteen records were collected along each shear wave line.

Data were collected using 4.5-Hz geophones connected to a Geometrics Geode seismograph which in turn was connected to a personal computer. The computer filtered, stacked, and recorded the signals. Stacking (adding) signals from multiple hammer blows at the same source point improves the signal to noise ratios of the recordings. Typically four recordings were stacked.

The program Surfseis developed by the Kansas Geological Survey was used to process the seismic records into S-wave profiles. From each seismic recording a fundamental-mode dispersion curve was extracted. The dispersion curve is related to the shear wave velocities of the different wave lengths contained in the surface wave. Longer wave lengths are related to the S-wave velocity of deeper soils and shorter wave lengths are related to the S-wave velocities of near surface soils. The dispersion curves are inverted into a series of one-dimensional S-wave velocity profiles that are concatenated together into a two-dimensional profile. More information of the MASW can be found at the Kansas Geological Survey's web site at www.kgs.edu/software/surfseis/.

III RESULTS

A. Downhole Results

Drawing 4 and Table 1 give the results of the two downhole P- and S-wave profiles collected in the test well. The two graphs show plots of P-and S-wave arrival times versus depth. Drawing 2 also shows the average P- and S-wave velocity for the upper 75 feet of soil along with the average small-strain shear modulus and small strain Poisson's ratio. The unit weight of the soil was assumed to be 110 pounds per cubic foot for calculating the shear modulus.

Table 1. Summary of Downhole Results

Layer Number	Depth (feet)	S-wave (fps)	P-wave (fps)
1	0 to 10	1100 to 1200	1900 to 2100
2	10 to 25	1300	2700 to 2800
3	25 to 40	800 to 850	1450 to 1500
4	40+	1000 to 1100	2400 to 3400

The data indicated four layers that were distinguished by their P- and S-wave velocities. Typically P- and S-wave velocities increase with depth. At the well site the second layer had higher S-wave velocities than the third layer and had the greatest S-wave velocity of all four layers. The higher S-wave velocity in the second layer may be due to weak cementing.

B. MASW Results

The results of the MASW data are shown on Drawing 5 and Table 2. Drawing 5 illustrates the S-wave velocity profiles collected along the four MASW lines and Table 2 shows the average S-wave velocities for each of the four seismic layers beneath each line along with an error estimate equal to one standard deviation.

Table 1. Average S-Wave Velocities for MASW Profiles

Line Number	Layer 1 S-wave (fps)	Layer 2 S-wave (fps)	Layer 3 S-wave (fps)	Layer 4 S-wave (fps)
Test Line	800	1650	700	1400
Sw-1	1000 ±240	1750 ±270	700 ±64	1450 ±150
Sw-2	850 ±77	1800 ±190	750 ±100	1200 ±150
Sw-3	1050 ±240	1600 ±270	750 ±73	1450 ±280

Layer	Depth (feet)
1	0 to 10
2	10 to 25
3	25 to 45
4	45+

The MASW data shows four seismic layers defined by their S-wave velocities (Drawing 5). Like the downhole data the MASW results indicate the second layer had a greater S-wave velocity than the third and had the highest S-wave velocity of all four layers. The higher velocities in the second layer may be from weak cementing.

Comparing the MASW data and the downhole data indicates the MASW tends to overestimate the velocities of the faster layers and to underestimate the velocities of the slower layers by about 20 percent. The S-wave velocities of layers 1 and 3, layers with low S-wave velocities, are probably not slower than the averages shown on Table 1. The S-wave velocities for layers 2 and 4, layers with high S-wave velocities, are probably not faster than the average shown on Table 1.

C. Near Surface Refraction Results

Along with the MASW data we collected a short refraction line at each shear wave profile. The results of the refraction lines are shown in Drawing 6. The refraction data indicated two layers in the upper 20 feet of soil. The first layer is only a few feet thick and probably consists of loose surface soils. The second layer had a higher P-wave velocity and consists of denser soils. The relatively high P-wave velocities found along lines Sw-1 and Sw-3 indicate a possible caliche layer.

D. Compressibility and Liquefaction

The S-wave velocities of the third seismic layer indicate a layer of soil that is likely to be weaker than the layers above and below it. The S-wave velocities measured for the third seismic layer at a depth ranging from 25 to 45 feet varied from 700 to 850 fps and were considerably slower than the S-wave velocities measured at other depths. We recommend testing this zone further with standard geotechnical methods.

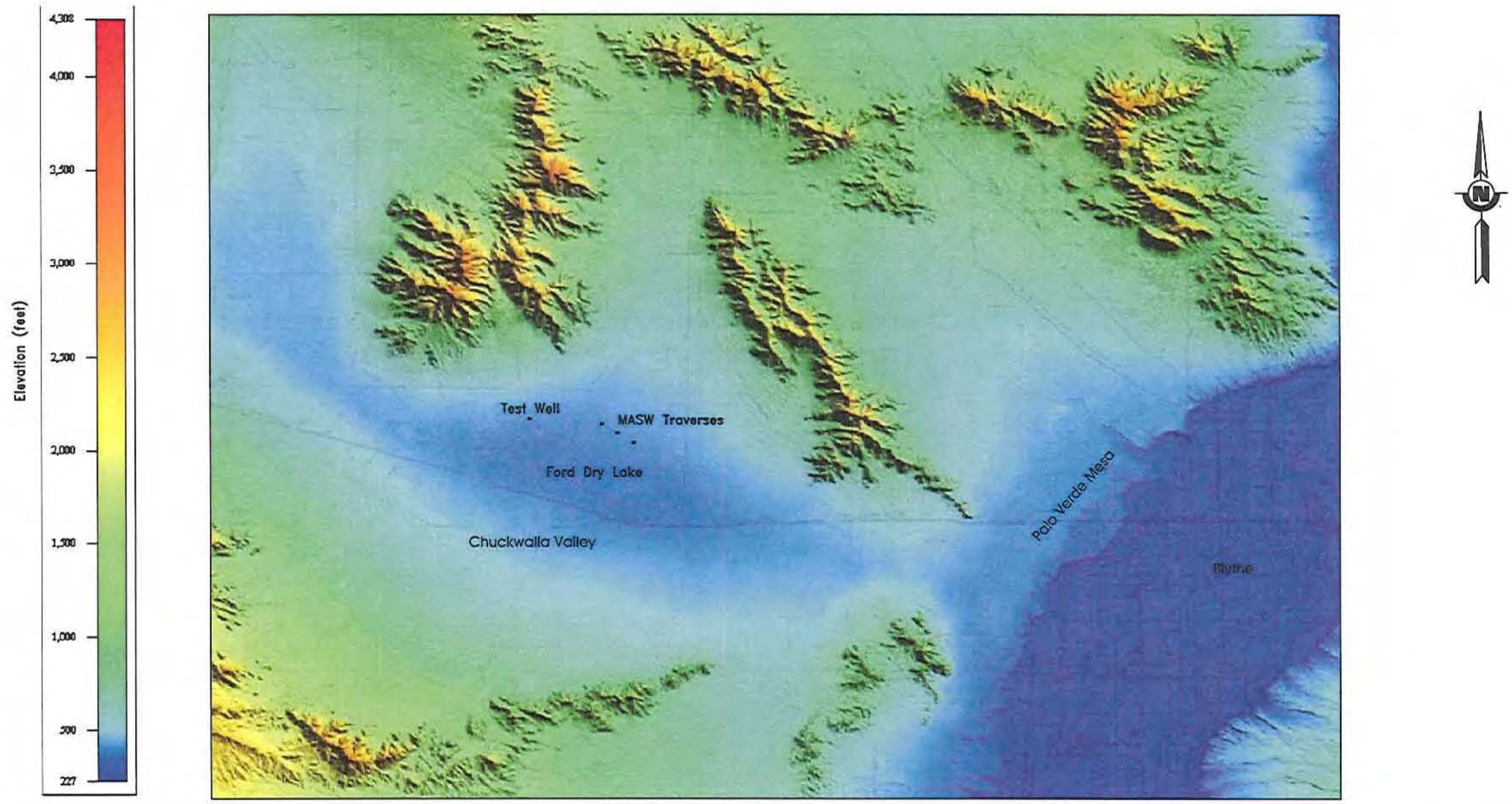
E. Summary

S-wave data were collected at four locations at the Ford Dry Lake site using two seismic methods (Drawing 3). Downhole shear wave data were collected at an observation well and shear wave velocity profiles were collected along four traverses using the multichannel analysis of surface waves method. Four seismic layers were found in the upper 75 feet of soil. The layers were distinguished by their P- and S-wave velocities (Drawings 4 and 5). The first layer was up to 10 feet thick and probably consisted of loose near surface soils. The second layer was between 10 and 25 feet below the surface. It probably consisted of denser sands, silts, and clays, possible lightly cemented, and may include caliche under lines Sw-1 and Sw-3. The third layer was from 25 to 45 feet below the ground surface. It was distinguished by P- and S-wave velocities that were slower than the soils above or below. The lower P- and S-wave velocities indicate the third layer was probably weaker than the layers above and below. We recommend testing the third layer further with conventional geotechnical methods such as a cone penetrometer or a standard penetration test. The fourth seismic layer probably consisted of denser sands, silts, and clays.

F. Limitations

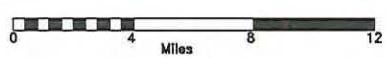
Seismic layers do not always correspond directly to lithologic changes that might be found in borehole or trenching data. A seismic layer is an interface between materials with different seismic wave velocities. Factors such as weathering, cementation, induration, and saturation as well as lithologic changes can create changes in seismic velocities. Also, there can be lithologic changes without velocity changes. However, our field experience indicates that seismic layers often correspond to changes in lithology, cementation, or saturation to within ±20% of the depth to the interface.

IV DRAWINGS



Explanation:

Shear Wave Velocity Test Location



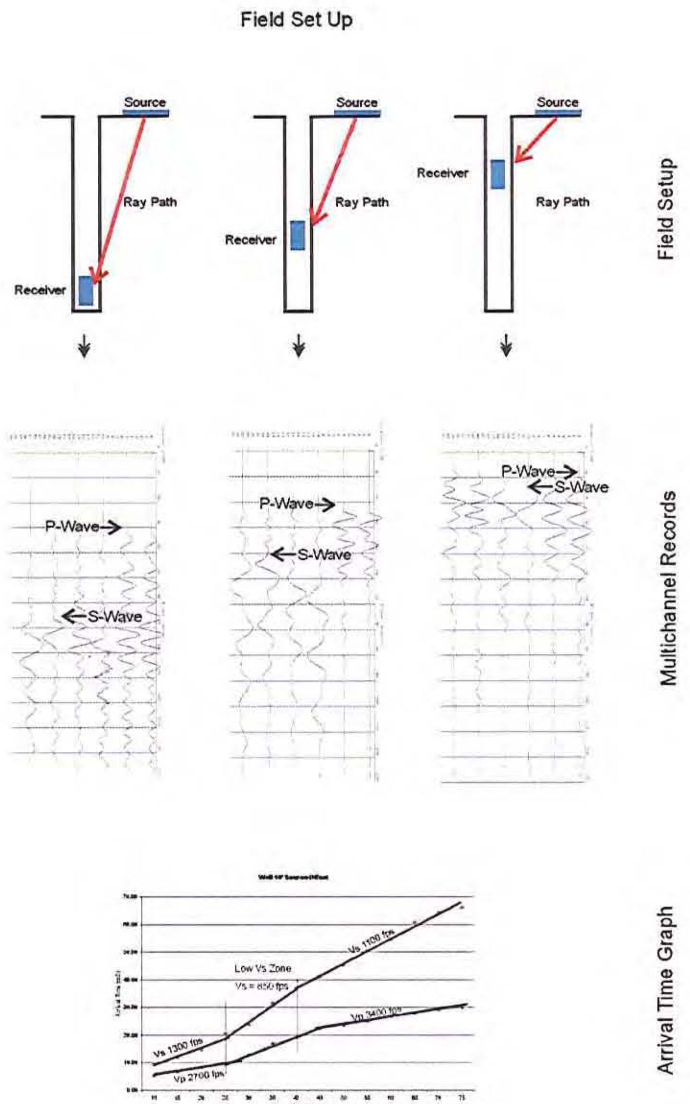
Elevation data from United States Elevation Data, NED, 30m Resolution

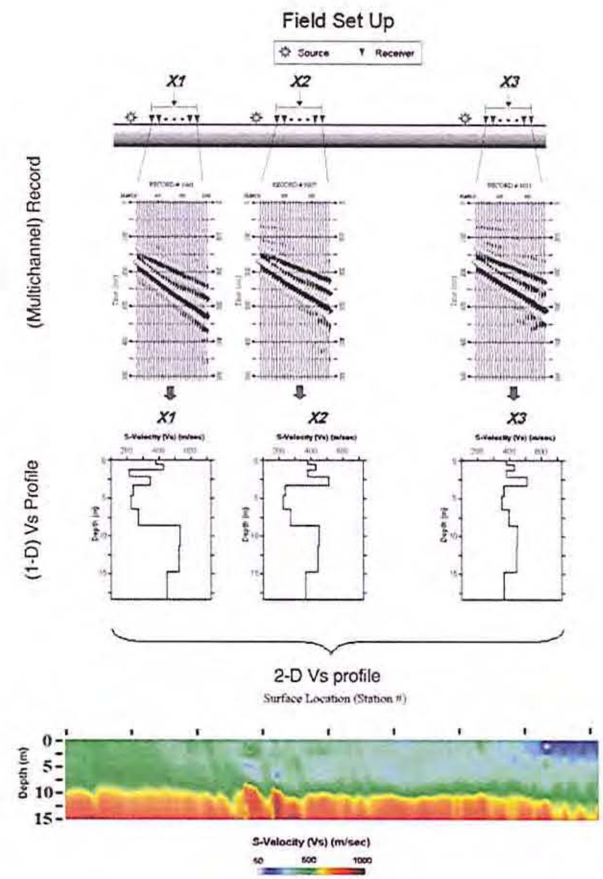
Vicinity and Elevation Map
Genesis Project Shear Wave Investigation
Riverside County California

SCALE: 1" = 4 Miles		DRAWN BY:	J.J.R.
DATE: 9-17-2009	JOB NUMBER: 129-263-09	REVISED:	

J R Associates Civil and Environmental Geophysics
1886 Emory Street, San Jose, CA (408) 293-7390

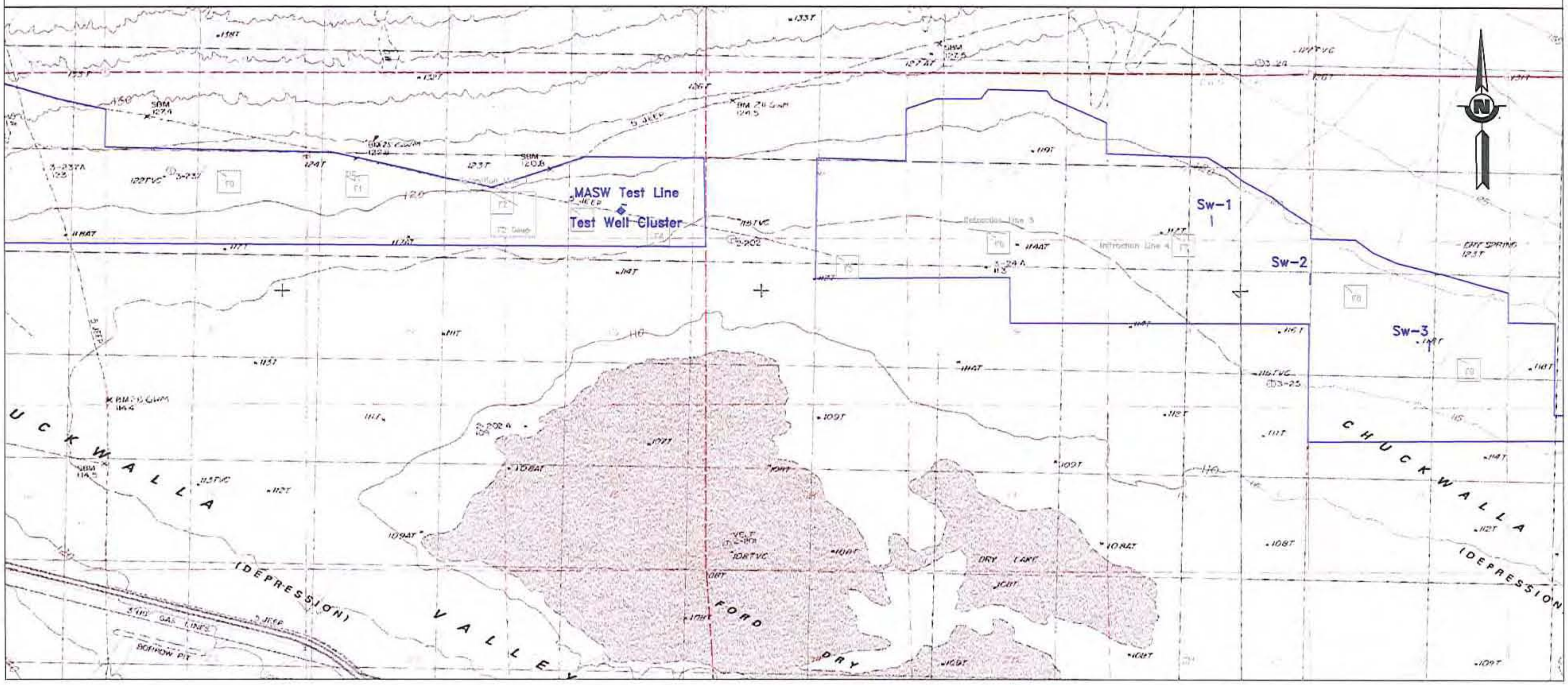
DRAWING NUMBER: 1





MASW graphic from the
Kansas Geological Survey Website

Downhole and MASW Shear Wave Techniques Genesis Project Shear Wave Investigation Riverside County, California			
SCALE: No Scale		DRAWN BY: J.J.R.	
DATE: 9-17-2009	JOB NUMBER: 129-263-09	REVISED:	
J R ASSOCIATES Civil and Environmental Geophysics 1886 Emory Street, San Jose, CA (408) 293-7390			
		DRAWING NUMBER: 2	



Explanation:

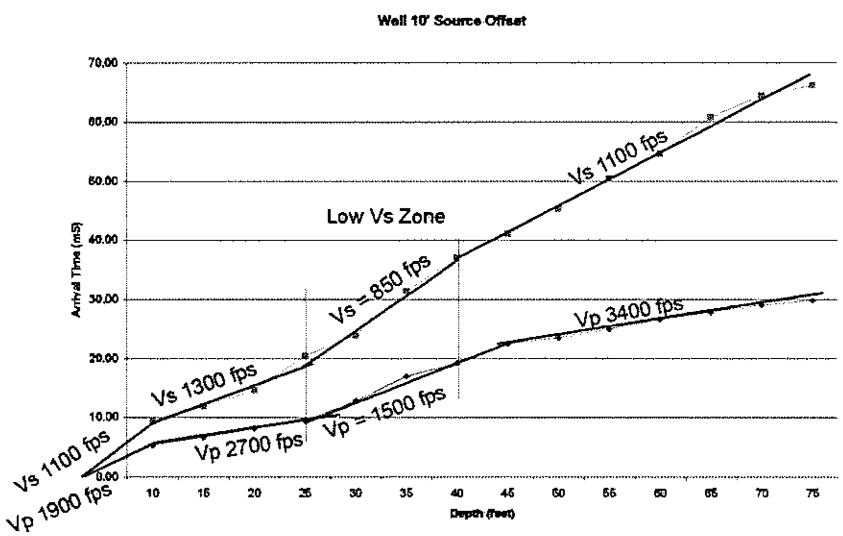
Sw-1 —— Shear Wave Velocity Profile

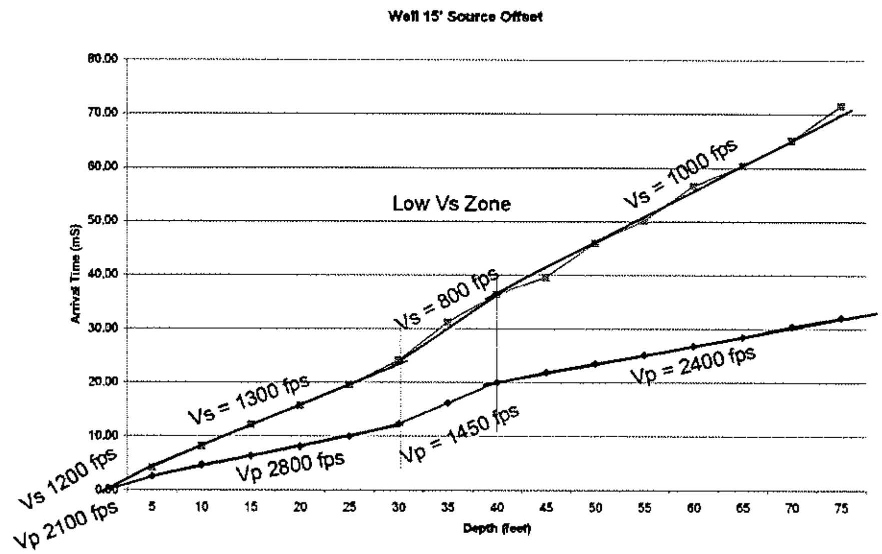
Downhole Shear Wave Measurements

0 0.5 1 1.5
Miles

Shear Wave Profile Lines Genesis Project Shear Wave Investigation Riverside County California			
SCALE: 1" = 0.5 Miles			DRAWN BY: J.J.R.
DATE: 9-17-2009	JOB NUMBER:	129-263-09	REVISED:
J R Associates Civil and Environmental Geophysics 1886 Emory Street, San Jose, CA (408) 293-7390			
			DRAWING NUMBER: 3

Downhole Shear Wave Measurements





Average Vp = 2300 fps
Average Vs = 1100 fps
Average Dynamic Shear Modulus $\rho Vs^2 = 2.9 \times 10^4$ psi
Average Dynamic Poisson's Ratio = .35
Unit Weight Assumed to = 110 lb/ft^3

Average Vp = 2200 fps
Average Vs = 1000 fps
Average Dynamic Shear Modulus $\rho Vs^2 = 2.4 \times 10^4$ psi
Average Dynamic Poisson's Ratio = .37
Unit Weight Assumed to = 110 lb/ft^3

Downhole P- and S-wave Arrival Times
Genesis Project Shear Wave Investigation
Riverside County, California

SCALE: See Diagrams		DRAWN BY: J.J.R.	
DATE: 9-17-2009	JOB NUMBER: 129-263-09	REVISED:	

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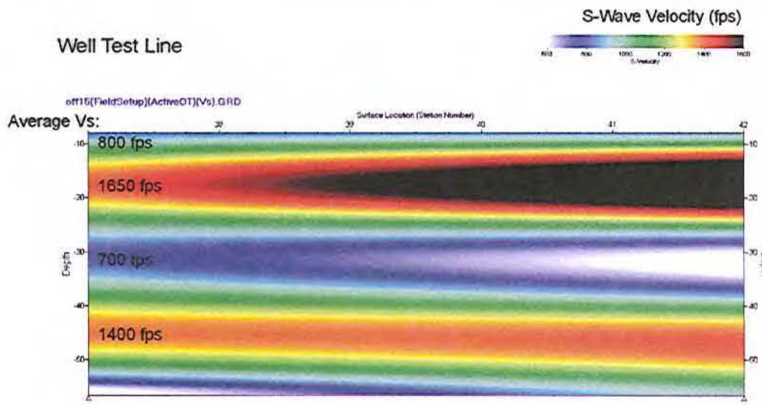
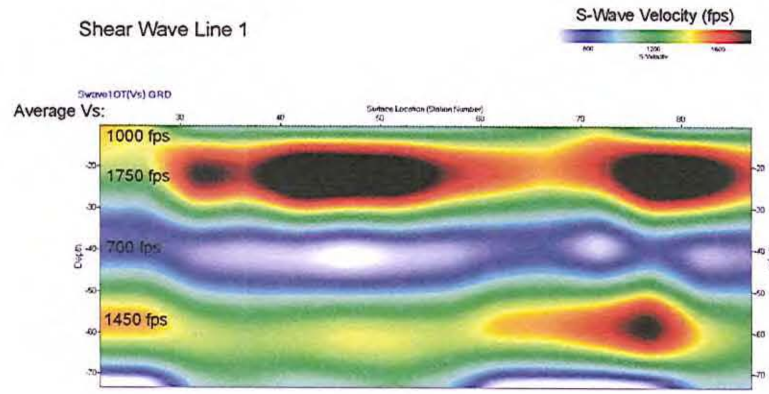
DRAWING NUMBER: 4

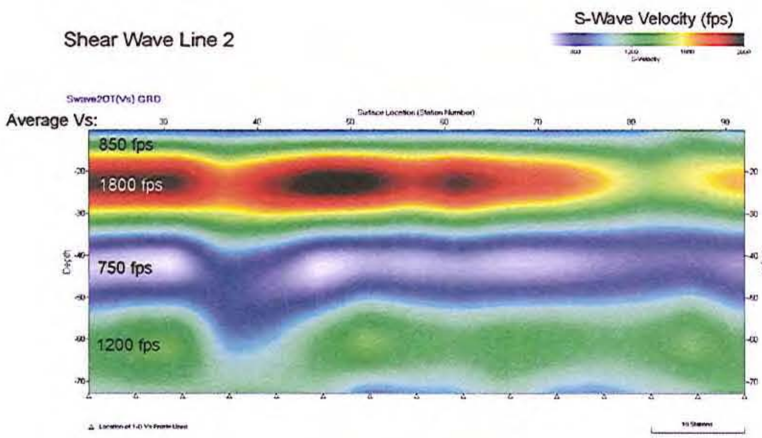
Well Test Line



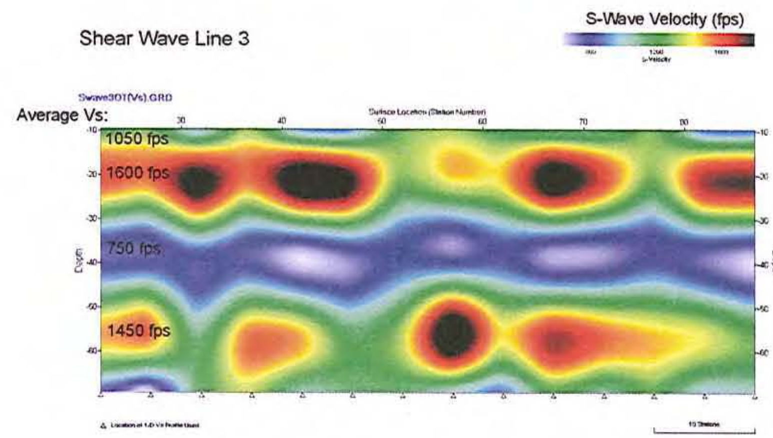
Shear Wave Line 1



Shear Wave Line 2



Shear Wave Line 3



Shear Wave Velocity Profiles Genesis Project Geophysical Investigation Riverside County, California			
SCALE: See Diagrams			DRAWN BY: J.J.R.
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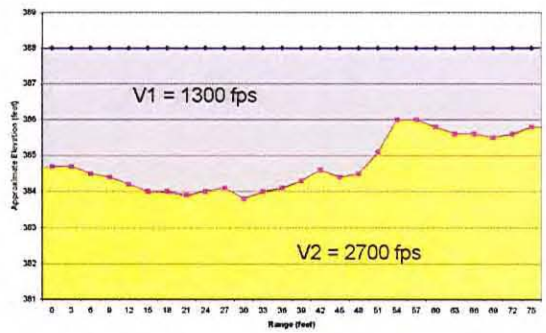
			DRAWING NUMBER: 5

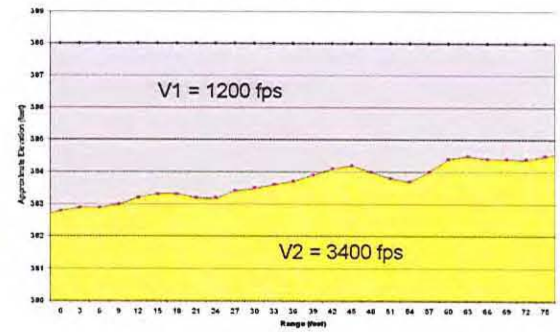
Refraction Line Near Test Well

Test Line 1



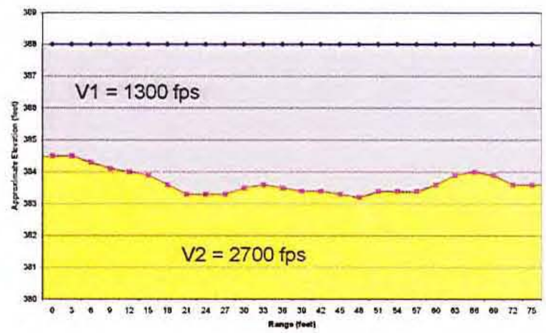
Sw-1 Refraction Profile

Shear Line 1- P-Wave Refraction Profile



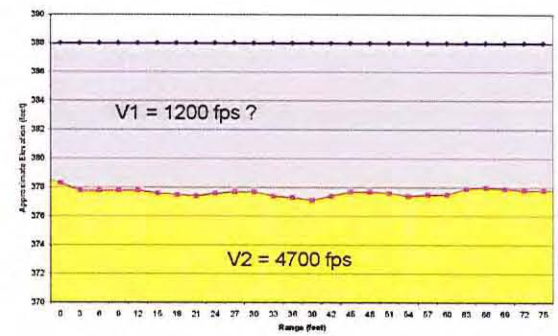
Sw-2 Refraction Profile

Shear Line 2- P-Wave Refraction Profile



Sw-3 Refraction Profile

Shear Line 3- P-Wave Refraction Profile



Note: Surface elevations were obtained from a USGS Topographic map of the area obtained through TerraServer-USA.com

Refraction Profiles Along Shear Wave Lines Genesis Project Shear Wave Investigation Riverside County, California			
SCALE: See Diagrams			DRAWN BY: J.J.R.
DATE: 9-17-2009	JOB NUMBER:	129-263-09	REVISED:
J R ASSOCIATES Civil and Environmental Geophysics 1886 Emory Street, San Jose, CA (408) 293-7390			
			DRAWING NUMBER: 6